In this Issue: Tests of Model Columns Duplicating the Quebec Bridge Chord Members.—The Design of Buttressed Dams.—Flooring vs. Stringers.—Test of a Screw Pump for River Flushing at Milwaukee.

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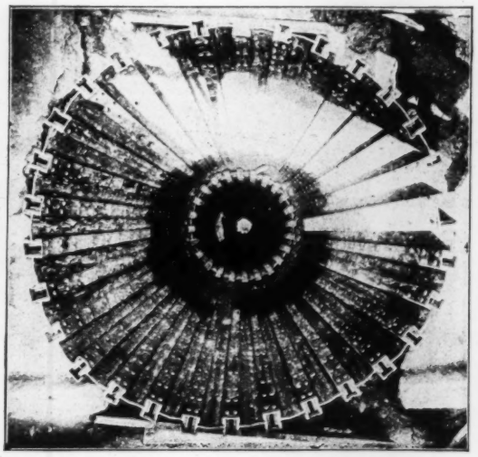

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A GERMAN RAILWAY VIADUCT OF CANTILEVER CONSTRUCTION WITH NOVEL HINGE DETAIL.

A single-track deck railway viaduct at Westerburg, Germany, on a new branch line of the Prussian State Railways, is of rather unusual design. Instead of having two-bent braced towers to support the several spans, it employs single rocker bents, and the truss system, instead of being either a continuous system or a series of simple spans, consists of a succession of cantilevers. The view, Fig. 1 herewith, shows the structure so clearly as to require little further explanation. This picture is taken from the "Zeitschrift für Bauwesen," where Regierungsrat Wolpert describes the viaduct.

The valley of the Hölzbach, which the viaduct crosses, is not deep, so that the highest bent is only 73 ft. high from top of masonry to bottom chord of trusses. The total length of the structure is 742 ft. between end shoes. The five spans are alternately 142 ft. and 158 ft. long, the middle and end spans being the short ones, while those containing the suspended spans have the greater length. The length of suspended span is 110.6 ft., or 7-10 of the length between centers of rocker bents, while each cantilever arm is 3-20 of the total span, or 23.7 ft. The depth of truss is constant, 15.8 ft. between gravity lines of chords. The trusses are spaced 10.8 ft. apart c. to c.

As the viaduct has no longitudinal stability in itself, it required to be anchored at the fixed end. Here an ordinary cast shoe is used for the vertical reaction, and a riveted steel anchor frame attached to the end of the bottom chord extends 41 ft. back into the masonry of the abutment. At the other end of the viaduct is a roller expansion bearing. There are, of course, no other expansion bearings in the bridge. Rail expansion is provided for by a miter slip joint in the rails over the expansion end of the viaduct.

The posts of the rocker bents have a batter of 1 in 6. At the foot they rest on round-topped cast shoes, and are anchored to the masonry against uplift; similarly the trusses are connected to the bents by flexible steel plates to resist any wind uplift. The posts are made up of latticed double-channel section, each channel composed of two 8 × 4 × 5/8-in. angles and a 14 × 7/8-in. plate. The lattice is 4 × 4 × 7-16-in. angle.

The truss members are of conventional cross-sections; the top chord is an inverted U-section of two channels with coverplate and two reinforcing angles in the re-entrant angles, while the bottom chord is made up of two channels with reinforcing web-plates where necessary, latticed top and bottom. The web members of the trusses are generally I-beams or four-angle I-sections. Connections are made with intersections on center-of-gravity lines.

The floor beams, riveted I-sections, rest on the top chord and are braced to it by plate-and-angle brackets on both sides. They cantilever out beyond the trusses, to carry a footwalk on either side of the track. I-beam stringers riveted between the floor beams (panel-length, 15.8 ft.) carry the ties. The track has outside guard timbers and inside guard rails consisting of inverted tees.

The most remarkable detail of the structure, undoubtedly, is the cantilever hinge, shown by Fig. 2. The hinges are midway between panel points, as the general view shows, and are at half depth of trusses instead of being in the line of either chord. As Fig. 2 clearly shows, the load at the hinge is transmitted by a cast shoe bearing set in the line of the compression diagonal, and this is held from slipping apart diagonally by a hinged tie from the adjacent top-chord panel point of the cantilever-arm. Should this tie fail, however, the shoe would still be held from slipping apart by the engagement of the overlapping edges of upper and lower shoe.

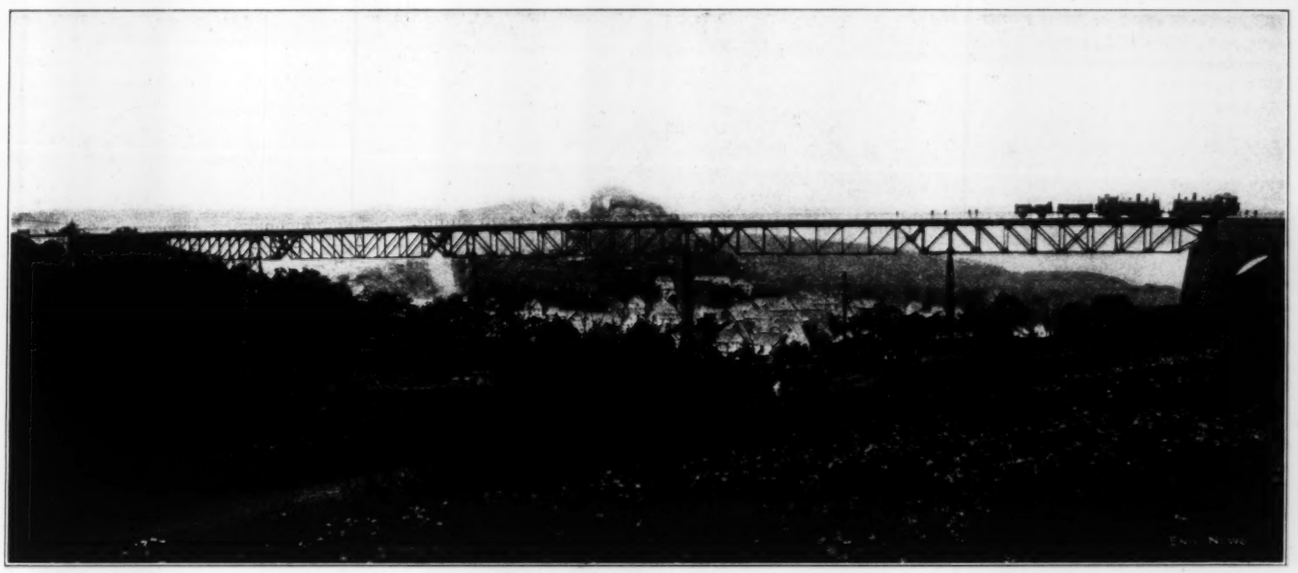


FIG. 1. VIADUCT AT WESTERBURG OVER THE HOELZBACH; PRUSSIAN STATE RAILWAYS.

The transfer of wind shear across the hinge is accomplished by a vertical pin in the center line of the bridge, at the transverse plane of the hinge, which pin is fixed in the terminal gusset plate of the wind truss of the cantilever-arm and engages a slotted hole in the end plate of the wind truss of the suspended span.

The design of this structure was worked out by the firm of Bruno Schulz, of Berlin, under the supervision of the construction department of the Frankfurt Division of the railway. The superstructure was built by the Tillmann Iron Co., of Düsseldorf, under the direction of its chief engineer, König.

Under the acceptance test, the maximum static deflections and the maximum dynamic deflections were:

Span. Deflections, Ins.	I.	II.	III.	IV.	V.
Static—Down	0.85	0.99	0.99	0.91	0.83
Up	0.12	....	....	....	0.10
Dynamic—Down	0.95	1.18	1.06	0.95	0.95
Up	0.16	....	....	....	0.16

Thus, the same test train gave deflections from 10% to 20% higher when moving rapidly than when at rest in the most unfavorable position.

The lateral sway under the test train was 0.24 in. maximum when the train moved slowly, while at high speed it was 0.40 in. at the bottom chord and 0.47 in. at the deck; measured in each case from extreme left-hand to extreme right-hand excursion.

Brake tests at ordinary running speed gave a maximum shortening of the entire structure of 0.10 in. when the train was moving toward the fixed end, and a maximum elongation of 0.06 in. when the train was moving in the opposite direction.

FLOORING VS. STRINGERS.

By LOUIS ROSS.*

The stringers of a highway bridge must be designed not only for a uniformly distributed load but also for a concentrated load, usually a certain weight resting on four wheels. The concentrated load usually controls the design of the stringers. When the plane of the front and rear wheels is half way between two stringers, the plank floor carries one-half of the wheel load to each stringer; but when the wheels rest directly over one stringer, the loaded stringer will not, as is usually assumed, carry the whole load, because the flooring will distribute part of it to the stringers each side of the loaded one.

The proportion of the total load distributed by the flooring depends on the relative stiffness of flooring and stringers. If the stringers are very heavy and stiff, while the flooring is thin, the loaded stiffness will carry practically all of the load; on the other hand a very stiff flooring will distribute the load over one or more stringers on either side of the loaded one. The problem of finding the distribution of loads has occurred to the writer while designing a set of standard steel and timber bridges. He has been unable to find in a number of text-books a rational solution, and therefore offers this one. The problem occurs also in all floor and roof systems, in designing the tie and stringer system over a railway trestle, and in the case of many other engineering structures.

In Fig. 1 let

L = span of stringer, inches;
a = spacing of stringers, c. to. c., inches;
I = moment of inertia of stringer, inches;
I_t = moment of inertia of flooring, inches;
W = equivalent single wheel load, pounds;
Q = load borne by loaded stringer, always less than W;

$\frac{W-Q}{2}$ = load borne by each adjacent stringer;

k = deflection coefficient, in the formula $D = \frac{1}{k} \cdot \frac{WL^3}{EI}$, depending on the loading and support of beams.

Now, the deflection of the loaded stringer is caused by a force Q, on a span of L,

$$D_1 = \frac{Q L^3}{k E I} \quad \ldots\ldots (1)$$

Likewise the deflection of each adjacent stringer is caused by a force $\frac{W-Q}{2}$, on a span L,

$$D_2 = \frac{(W-Q) L^3}{2 k E I} \quad \ldots\ldots (2)$$

Also the deflection of the flooring is caused by a load $(W - Q)$, on a span of $2a$

$$D = \frac{(W-Q)(2a)^2}{k_t E_t I_t} \quad \ldots\ldots (3)$$

The subscript f refers to flooring.

But the deflection of the floor-plank is equal to the difference between the deflections of the loaded and adjacent stringers; hence

$$D = D_1 - D_2,$$

or

$$\frac{(W-Q)(2a)^3}{k_t E_t I_t} = \frac{Q L^3}{k E I} - \frac{(W-Q) L^3}{2 k E I} \quad \ldots\ldots (4)$$

whence

$$\frac{Q}{W} = \frac{1+n}{3+n} \quad \ldots\ldots (5)$$

where

$$n = 16 \frac{a^3 k E I}{L^3 k_t E_t I_t} \quad \ldots\ldots (6)$$

The expression for n, Eq. (6), is the measure of the relative stiffness of floor-plank and stringer. Thus the stiffness of any beam depends on its length, moment of inertia, modulus of elasticity and method of loading. The expression $\frac{1}{k} \cdot \frac{L^3}{EI}$ is a measure of stiffness of any beam; but since the stiffness varies inversely with this expression we may write

$$\frac{L^3}{k E I} = \frac{1}{\text{stiffness}} = \frac{1}{S} \quad \ldots\ldots (7)$$

and

$$n = 16 \frac{k E I}{L^3} \div \frac{k_t E_t I_t}{a^3} = 16 \frac{S}{S_t} \quad \ldots\ldots (8)$$

The curve in Fig. 2 shows to the eye how the percentage of the load, $100 \frac{Q}{W}$, borne by the loaded stringer varies with n, the relative stiffness of the stringer and floor-plank.

It is evident that each stringer must be designed to carry at least half of the concentrated wheel load W, for the case when the wheel comes half way between two stringers. Also, the floor-plank must be designed for both of the following conditions: (1) The whole load W, on a span a; and (2) the load $(W - Q)$ on a span $2a$. Economic design demands that the moments be equal, hence,

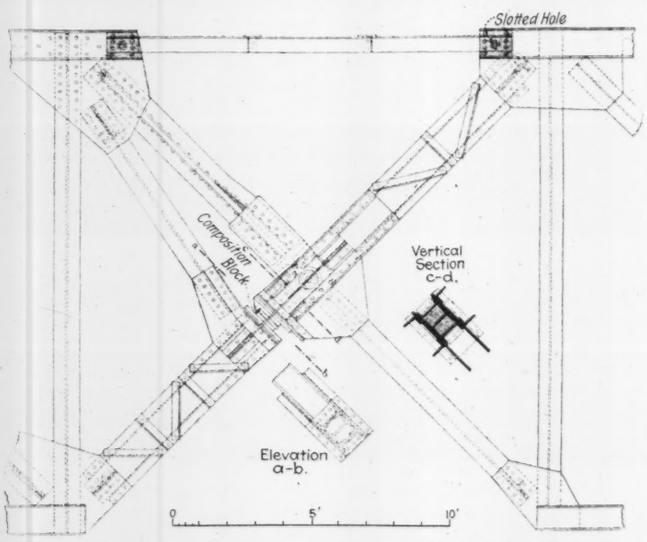


FIG. 2. HINGE AT CONNECTION OF SUSPENDED SPAN TO CANTILEVER ARM, WESTERBURG VIADUCT.

$$M = \frac{W a}{4} = \frac{(W-Q) 2a}{4} \quad \ldots\ldots (9)$$

whence

$$Q = \frac{W}{2} \quad \ldots\ldots (10)$$

Since the flooring must, according to Eq. (10), be strong enough to carry half of the load, it would be waste of material to design the stringer for carrying more than the other half of the load. The safe deduction, therefore, is:

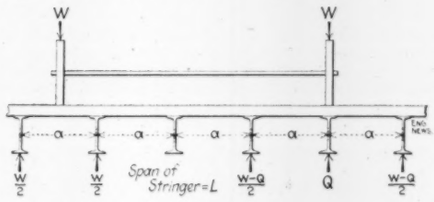


Fig.1.

The relative stiffness and strength of the stringer and flooring must be such that the stringer shall carry half of the concentrated load, and the floor-plank will distribute the other half, one-quarter to each adjacent stringer.

The writer believes this deduction to be original, and true within the limits of accuracy given at the end of this paper.

From Fig. 2 it is seen that the value of n corresponding to $\frac{Q}{W} = 50\%$ is $n = 1$, and from Eq. (8), substituting $n = 1$, we get

$$S_t = 16\, S \quad \ldots\ldots (11)$$

or

For economic design, the flooring must be 16 times as stiff as the stringer,

the floor stiffness being measured by $\frac{k_t E_t I_t}{a^3}$, and the stringer stiffness by $\frac{k E I}{L^3}$. Algebraically this is expressed by

$$\frac{L^3}{a^3} = 16 \frac{k E I}{k_t E_t I_t} \quad \ldots\ldots (12)$$

We may, therefore, for the sake of emphasis, combine the two deductions made above as follows:

If the flooring is 16 times as stiff as the stringer, then the maximum load that can come on either is 50% of the total; if the stringer is stiffer than this then it will carry more than 50%, the limiting value being 100%, and the maximum load on flooring will still be 50%; if the stringer is relatively not stiff enough, then the flooring will have to carry more than 50%, the limiting value being 67%, over a span of $2a$, while the maximum load on the stringer will still be 50% for the case when the load lies halfway between two stringers.

Of the equations stated above the working equations are (5), (6) and (12). Eqs. (5) and (6) are used to determine the stresses in an existing structure or in one already designed. The value of n is found from Eq. (6) or from Fig. 3; then the percentage, $100 \frac{Q}{W}$, is found from Eq. (5) or from Fig. 2. After the loads are thus found the stresses are computed by the usual methods. Fig. 3 has been plotted to facilitate the solution of Eq. (6) for n, in the case where $E = E_t$, that is when the materials of flooring and stringer are the same, as in the case of a plank-floor over stringers of wood, or corrugated iron or steel trough floor over steel

*111 Montgomery St., San Francisco. Ross & Brett, Engineers. U. S. Deputy Surveyor.

beams, and when also the k's or methods of support are the same. The curves then give the solution of the equation:

$$n = 16 \frac{a^3}{L^3} \frac{I}{I_t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6a)$$

The scale of ordinates on the right hand side of Fig. 3 gives likewise curves for solving Eq. (6) in the case of a wood floor on steel beams. The value of E has been taken at 30,000,000 lbs. per sq. in. for steel and 600,000 lbs. per sq. in.

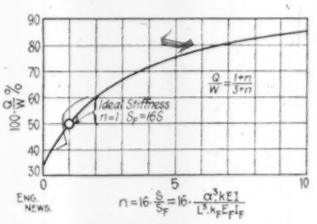


Fig. 2.

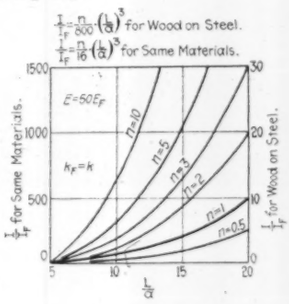


Fig. 3.

for timber, that is for the case when $E = 50\ E_f$; this latter contains a factor of safety of 2 to allow for variation in the quality of the wood, and is the average of the values recommended for the usual woods by the Committee on Strength of Bridge and Trestle Timbers of the Association of Railway Superintendents of Bridges and Buildings. The right hand ordinates then give the solution of the equation:

$$n = 800 \frac{a^3}{L^3} \frac{I}{I_t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6b)$$

The curves give the relation between $\frac{I}{I_t}$, $\frac{L}{a}$ and n for the case when $k = k_t$. For other cases the values may be obtained by proportion.

To design a floor system the relation between $\frac{L}{a}$ and $\frac{I}{I_t}$ is first determined either from Eq. (12) or from the curve $n = 1$ in Fig. 3. For the case of similar supports and similar material Eq. (12) becomes

$$\frac{L^3}{a^3} = 16 \frac{I}{I_t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (12a)$$

and for steel beams with wood flooring

$$\frac{L^3}{a^3} = 800 \frac{I}{I_t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (12b)$$

If this equation is satisfied, then the stringer need have a modulus section for half the load only, and the flooring for half the load over a span equal to twice the stringer spacing.

The values of k for a beam carrying a concentrated load at the middle are: 48 when the beam is simply supported, and 192 when fixed at the ends. If the concentrated load is carried on a broad tire, say 12 ins. wide, while the stringer spacing is, say, 24 ins., then the k for the flooring is between 48 and 77, the latter being the value of k for a distributed load, with supported ends. The method for determining the exact value of k for such cases can be found in most text-books, and is outside the range of this article. The action of floor spikes in ordinary wooden floors cannot be assumed to cause the flooring to act as a fixed beam, their action being too uncertain; the value of k_t therefore for flooring is 48, and in the great majority of cases the stringers are also simple beams, giving $k = 48$.

The only assumption made in the development of this theory is that only three stringers carry the load, not five or seven. Theoretically, the flooring will distribute the load partly over all the stringers it can reach; but rough computation shows that for ordinary cases the second stringer either side of the loaded one carries no more than 3% of the whole load. The neglect of this small amount simplifies the computation, and is on the side of safety. The uncertainties in the k's and E's fully justify the omission of this minute refinement.

The wear of the planks may be allowed for in the design, though the factor of safety of 5 or 6 in stress and 2 in the modulus of elasticity takes care of this.

Ordinarily the design of a beam consists in selecting a suitable section modulus, $\frac{I}{c}$, to carry the required bending moment. Then the designer is left to his own choice in proportioning the width to the depth of the member. Eq. (12) gives the true theoretic condition for obtaining this ratio of width to depth. The designer must, of course, not neglect other factors controlling the selection, such as market sizes, distributed loads, minimum sections, etc.

In the following examples two improperly designed floors, i. e., improper for concentrated loads only, have been assumed, the faultiness proved, and the design changed, showing the resulting economy.

EXAMPLE 1.—Consider a bridge floor consisting of 10-in. 25-lb. I-beams, 12 ft. long, spaced 18 ins. c. to c., with a 3 × 12-in. flooring. From Eq. (6b) or Fig. 3 we find $n = 7.07$, and from Eq. (5) or Fig. 2, $Q = 0.80\ W$. Thus the stringers carry 80% of the load while the planks must be strong enough to carry the whole load on a span of 18 ins. This shows that the stringer is relatively too stiff. Substituting a 9-in. 21-lb. I-beam with the same spacing, and a 4 × 12-in. flooring, we get $n = 2.08$, and $Q = 0.61\ W$. The section modulus of the 9-in. I-beam is, of course, less than that of the 10-in. beam in the ratio of $\frac{18.9}{24.4} = 0.775$; but the fraction of the whole load that the 9-in. beam has to carry is only $\frac{61}{80} = 0.76$ as great as that for the 10-in. beam. Hence the 9-in. beam will carry a slightly greater load than the 10-in. beam if the planking is made heavier. The added amount of lumber for one stringer length is 18 ft. B. M., while the saving of steel is 48 lbs. This shows the great saving effected by applying this method of design.

EXAMPLE 2.—Consider the case of a floor composed of wooden stringers 14 ft. long, and wooden flooring on top, to carry a concentrated load of 5,000 lbs. The flooring will be made stiff enough so that the loaded stringer will not carry more than half of the load. The required $b\,d^2$ for the stringer, with a safe unit stress of 1,000 lbs. per sq. in., is 640 ins.^{3}. This is based on a 2,500-lb. load. The required $b_t\,d_t^2$ for the flooring is 7.5 a where a is the spacing of stringers, to be determined; this is based on a load of 5,000 lbs. over a span a, or 2,500 lbs. over a span 2 a. [See Eq. (9) and (10).] Also from Eq. (12a) we find

$$a = \frac{L}{2} \frac{d_t}{d} \sqrt[3]{\frac{b_t}{2\,b}} = 84 \frac{d_t}{d} \sqrt[3]{\frac{b_t}{2\,b}}$$

We have then the following three conditions to satisfy as closely as market sizes will allow:

(a) $bd^2 = 640$

(b) $b_t\,d_t = 7.5\ a$

(c) $a = 84 \frac{d_t}{d} \sqrt[3]{\frac{b_t}{2\,b}}$

A 5 × 12-in. stringer satisfies the first condition. A 3 × 12-in. floor-plank in condition (b) requires the spacing to be about 15 ins. Condition (c) then gives the spacing to be 22 ins. This shows that with a 15-in. spacing the floor-plank is too stiff, and would therefore have to transmit more than 2,500 lbs. on a span of 30 ins., which it cannot safely do. On the other hand a spacing of 22 ins. would leave the plank only 2,500 lbs. to carry, but it could not do it safely as shown by condition (b). The remedy then is to use thicker flooring with a larger spacing of stringers. Thus, using a 4 × 12-in. flooring we get from condition (b) $a = 25.6$ ins., while condition (c) gives $a = 23.6$ ins. Spacing 24 ins., and 4 × 12-in. flooring, with 5 × 12-in. stringers, therefore, makes a satisfactory system. The added lumber for the flooring for a width of 2 ft. and a length of 14 ft. is 28 ft. B. M., while the saving effected by the wider spacing of the stringers is 42 ft. B. M., and the structure is stronger.

In order to show how the current practice agrees with that suggested in this paper, the following table has been computed, to show the existing values of n and $100 \frac{Q}{W}$ in highway bridges. These examples were taken from standard designs of highway bridges made by the writer for the U. S. Reclamation Service some time ago. The loads and all pertinent data are given in the table.

A study of this table shows that the usual timber stringer has about the right stiffness; while almost all the steel stringers are far too stiff in comparison with the flooring. Had the modulus of elasticity for wood been taken at its full value, 1,200,000, then the values of n would have been halved.

The above analysis was made on the assumption

TABLE OF n AND $\frac{Q}{W}$ FOR VARIOUS SPANS AND SPACINGS.

L span ft. ins.	a spacing.	Stringer size.	Flooring size.	n.	$100 \frac{Q}{W}$	REMARKS.
8-8	24 ins. c. to c.	3 x 10	3 in. x 12 in. flooring throughout.	1.8	58	3 and 4 panel Pratt truss highway bridges, 26 to 52 feet span; to carry usual dead load, and 80 pounds per square foot live load, and an 8-ton roller for a concentrated load. Safe unit-stress in wood 1,000 pounds per square inch, tension or compression.
9-4		3 x 12		2.5	63	
11-4		3 x 12		1.4	55	
12-8		4 x 12		1.4	55	
14-0		"		1.0	50	
12-0		"		1.6	56	
12-6		"		1.4	54	
13-0		"		1.2	52	
8-0		3 x 10		2.3	62	Plain wooden stringer bridges, 16 ft. wide, to carry a 10,000-lb. concentrated load on four wheels. All other conditions the same as above.
10-0		3 x 12		2.0	60	
12-0		4 x 12		1.6	56	
16-0		4 x 14		1.0	50	
20-0		5 x 16		1.0	50	
24-0		5 x 18		0.8	47	
28-0		5 x 20		0.7	46	
30-0		6 x 20		0.7	46	
13-3	18 c. to c.	10"-25 lb. I.	3 in. x 12 in. flooring throughout.	5.0	75	Steel truss bridges of 3 and 4 panels for the same conditions as preceding type. Safe stress in steel 16,000 lbs. per sq. in. The concentrated load here was 18 tons.
13-0		"		5.5	76	
14-0		"		4.5	73	
15-0		"		3.6	70	
16-0		"		3.0	67	
25-0		15"-42 lb. I.		2.8	65	Steel stringers, 18 tons concentrated load.
11-8	30 inches c. to c.	5"- 9.75 lb. I.	3 in. x 12 in. flooring throughout.	3.5	70	Steel trusses of 3, 4, 5 and 6 panels. Concentrated load of 10,000 lbs. on four wheels.
13-3		6"-12.25 " I.		4.3	73	
16-8		7"-15. " I.		3.6	70	
13-9		6"-12.25 " I.		3.9	71	
15-0		"		3.0	67	
13-0		"		4.6	74	
14-0		"		3.7	70	
16-0		7"-15. " I.		4.1	72	
18-0		"		2.9	66	
16-8		"		3.6	70	

that the loaded plank carries all of the load $(W - Q)$, which is distributed to the adjacent unloaded stringers. As a matter of fact the planks either side of the loaded one will relieve the loaded plank somewhat—the amount depending on the rigidity with which the plank floor is attached to the stringers. A very slight separation between the flooring and stringer will vitiate the load carrying capacity of the unloaded planks. Thus, take the case of 3 × 12-in. flooring, with stringers 18 ins. c. to c., and a concentrated load of 10,000 lbs. From Eq. (3), taking Q equal to ½ W, we find that the deflection of the loaded plank across the middle of the stringer is 0.15-in. The adjacent planks will, of course, have a deflection considerably less than this. Any ordinary method of attachment, such as bolts, spikes, or clips, will after a short wear allow a play of at least 0.10-in., caused by the rust, wear and moisture under the head of the spike or bolt, allowing the wooden floor to separate from the stringer. Therefore, in a wooden floor no reliance should be placed on the help of the unloaded planks.

In the limiting case, however, where the flooring is continuous over the stringers the distribution of the load requires further investigation. The following analysis is made for a reinforced-concrete floor system. With slight changes in the value of k, the deflection coefficient, and in the formula for the deflection curve, this analysis can be easily adapted by the reader to apply to any continuous floor system.

Fig. 4 shows a typical reinforced-concrete floor system, carried on columns, "girders," and subsidiary beams, called here "stringers." The reinforcement is assumed to be so placed and bent over supports that the stringers and floors are truly continuous. This assumption is made merely to define the problem; any other degree of continuity may be assumed. The girders will be taken as absolutely unyielding; and the slab will be reinforced in both directions, so as to be able to transmit stresses not only across the stringers but also along them. A concentrated load W is placed over the middle of one of the stringers; the question is what part of the load is taken up by the floor-slab and transmitted to the adjacent unloaded stringers.

Using the same notation as before, Eq. (1) and (2) apply without change. The slab, however, must be treated differently, since its effective width, hence its moment of inertia is unknown.

The greatest deflection of the slab is under the load, the deflection growing less and less toward the girders, where it is practically nothing. This deflection distributes the load $(W - Q)$ along the whole length L of the stringer. Conceive the slab to be divided into a number of strips, each of width dx and length $2a$; the strip $n - n$ being a typical one. Each strip forms a beam, fixed at the ends by the continuity of the reinforcement, and loaded at the middle. Since the stiffness of any two strips is equal, because their length, depth, width and method of support are equal, the load borne by any two strips is proportional to the deflections at the center of each strip, which is identical with the difference of deflections of the loaded and adjacent stringers at the given strips.

The equation of the flexure curve of the stringer, considered as a continuous beam, carrying a load Q at its middle is:

$$y = \frac{Q}{48\,E\,I}\,x^2\,(3\,L - 4\,x) \qquad \ldots\ldots(13)$$

This equation can be deduced from the moment equation of a fixed beam, as given in most textbooks on mechanics. The deflection at the middle of the stringer is:

$$D_1 = \frac{Q\,L^3}{192\,E\,I} \qquad \ldots\ldots(14)$$

Let A be the load carried by one inch width of slab at the middle of the stringer. Then the load on a width dx is $A\,dx$ pounds; and the load on the strip $n - n$, being proportional to the deflection, is:

$$dP = A\,dx\,\frac{Q}{48\,E\,I}\,x^2\,(3\,L - 4\,x) \div \frac{Q\,L^3}{192\,E\,I} = \frac{4\,A}{L^3}\,x^2\,(3\,L - 4\,x)\,dx \qquad \ldots\ldots(15)$$

The sum of all the infinitesimal loads carried by all the strips of half of the slab, between the concentrated load and the girder is ½ $(W - Q)$. We may therefore write:

$$\frac{1}{2}\,(W - Q) = \frac{4\,A}{L^3}\int_0^{\frac{L}{2}} x^2\,(3\,L - 4\,x)\,dx = \frac{4\,A}{L^3}\cdot\frac{L^4}{16} = \frac{A\,L}{4} \qquad \ldots\ldots(16)$$

Hence

$$A = 2\,\frac{W - Q}{L} \qquad \ldots\ldots(17)$$

The average intensity of the load along the stringer is

$$\frac{W - Q}{L}$$

Eq. (17) shows that the intensity of the load at the middle strip is twice as great as the average intensity.

The intensity of load distribution along the stringer is shown by the series of horizontal lines on the plan of Fig. 4. These have been made proportional to the ordinates of the deflection

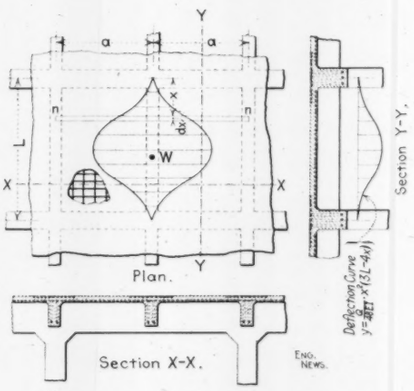


Fig. 4.

curve, which is shown on the horizontal section.

Knowing the load, we find the deflection of the slab at the middle to be:

$$D = \frac{2\,(W - Q)}{L}\cdot\frac{(2\,a)^3}{k_t\,E_t\,I_t} \qquad \ldots\ldots(18)$$

The notation is the same as before, with the exception that I_t now stands for the moment of inertia of a 1-in. width of the slab. Following through the same process of reasoning as in the case of a wooden floor, we have:

$$D = D_1 - D_2$$

or

$$\frac{2\,(W - Q)(2\,a)^3}{L\,k_t\,E_t\,I_t} = \frac{Q\,L^3}{K\,E\,I} - \frac{(W - Q)\,L^3}{2\,k\,E\,I} \qquad \ldots\ldots(19)$$

whence

$$\frac{Q}{W} = \frac{1 + n}{3 + n} \qquad \ldots\ldots(5)$$

in which

$$n = \frac{32\,k\,E\,I}{L\,L^3} \div \frac{k_t\,E_t\,I_t}{a^3} = \frac{32}{L}\cdot\frac{S}{S_t} \qquad \ldots\ldots(20)$$

In applying Eq. (20) to reinforced concrete it must be noticed that $E = E_t$, provided the same quality of concrete is used for the slab as for the stringer; also that $k = k_t$, if the degree of continuity of the slab and stringer is the same. The moment of inertia, I, of a concrete beam is not ordinarily used in design, nor will any attempt be made here to derive a formula for this property of a reinforced-concrete beam. For the purpose of applying Eq. (20) it is sufficient to know the ratio of $I \div I_t$. A little consideration will show that, as in the case of rectangular wooden beams, the moment of inertia of two beams of the same quality of concrete with the same percentage of reinforcement is proportional to their values of $b\,d^3$, where

b = width of beam,
d = effective depth of beam.

That the moment of inertia is proportional to the width is self-evident, requiring no proof; its proportionality to the cube of the depth may be shown by analogy to the case of a wooden beam—the part of the beam above the neutral axis, which is in compression, follows the identical laws of a homogeneous beam, assuming linear distribution of stress. The steel in tension increases in area in proportion to the depth, if the percentage reinforcement is constant, and it also increases its unit stress as well as its moment arm in proportion to its distance from the neutral axis. Hence both the steel and the concrete increase their moments of inertia as the cube of the depth. Substituting $b\,d^3 = b_t\,d_t^3$ for $I = I_t$ in Eq. (20), and remembering that $b_t = 1$ in., we get:

$$n = 32\,\frac{b}{L}\left(\frac{d}{d_t}\right)^3\left(\frac{a}{L}\right)^3 \qquad \ldots\ldots(21)$$

As in the case of a plank floor, ideal economic design requires that the slab should be able to distribute half of the whole load, one-quarter to each of the adjacent beams. For this case $n = 1$, and Eq. (21) becomes

$$\left(\frac{a}{L}\right)^3 = \frac{L}{32\,b}\left(\frac{d_t}{d}\right)^3 \qquad \ldots\ldots(22)$$

Eq. 22 is used in designing a floor system, while Eqs. (21) and (5) will solve the distribution of load in any existing floor system. Thus, let the stringers be 10 × 16 ins., 10 ft. long, and spaced 5 ft. apart, while the flooring is 6 ins. thick, both depths being measured above the center of the steel reinforcement. From Eq. (21) we find

$$n = 32\,\frac{10}{120}\left(\frac{16}{6}\right)^3\left(\frac{5}{10}\right)^3 = 6.3$$

and from Eq. (5) we solve

$$Q = 0.78\,W.$$

That is the loaded stringer carries 78% of the whole load and each of the adjacent ones carries 11%. A better distribution can be effected by decreasing the spacing of the stringers, increasing the thickness of the slab, or by decreasing the stiffness of the stringer. Thus changing the effective depth of the stringer to 12 ins., the value of n becomes 2.1, and $Q = 0.60\,W$. The load-carrying capacity of the floor is thereby increased, even though the stringer has been actually weakened, by sending a greater part of the load to be carried by the adjacent stringers.

The amount of reinforcement parallel to the slab should be sufficient to carry the load ½ $(W - Q)$ from the center of the stringer to the girder. Thus, in the example quoted above, where Q is 60% of W, the longitudinal reinforcement should be capable of transmitting 20% of the total concentrated load.

If a uniformly distributed load is applied at the same time with the concentrated load, then the stresses due to each should be computed and added to obtain the total stress. Those due to the distributed load are independent of the stiffness of the various members, and are determined by the usual formulas, while those caused by the concentrated load are computed by the formulas given above, as these depend primarily on the stiffness of the various members.

THE PASSAIC RIVER WATER DISTRICT BILL, which would have enabled the New Jersey Water-Supply Commission to construct an immense reservoir at Mountain View, for water storage and flood prevention purposes, was withdrawn from the legislature just before adjournment, at the request of the Commission. The proposed dam, and reservoir scheme, and also the water district scheme were outlined in our issue of April 2, 1908, pp. 383 and 384, respectively.

REINFORCED-CONCRETE CANTILEVER GIRDERS IN THE BOYERTOWN BUILDING, PHILADELPHIA.

Since the erection of the 16-story, 210 ft. Ingalls building in Cincinnati in 1903, the high reinforced-concrete building has become so general a construction that its details have become nearly as standardized as the steel structure from which its form was modeled. This similarity of layout, shapes, sizes and reinforcement

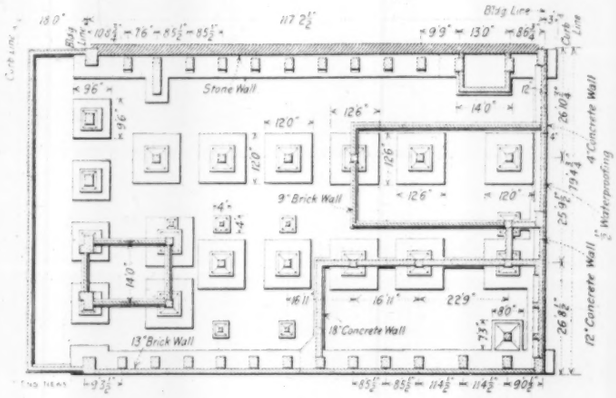


FIG. 1. FIRST FLOOR PLAN OF BOYERTOWN BUILDING, PHILADELPHIA.
Ballinger & Perrot, Architects and Engineers.

distribution is such that the description of any new work along this line is apt to be mere repetition and of interest only to those immediately connected with the building under discussion. However, in every new structure there is some detail which is novel either in conception or application and which is worthy of more extended notice than the general design itself would warrant.

On account of this standardization, a brief description of the main design of the new Boyertown Building, in Philadelphia, will suffice. The more important details will be taken up at more length. The building is located in the heart of the business district of Philadelphia, on a lot 79 × 117 ft. in area extending through from Arch St., between 12th and 13th Sts., to a narrow alley known as Appletree St. It has ten stories, a mezzanine floor and a basement and the top of the roof is 178 ft. above the street level. The usual reinforced-concrete construction has been employed, with granite, brick and terra cotta curtain face walls. There are two longitudinal rows of columns dividing the floor system into bays about 25 × 17 ft. in plan. (Fig. 1.) All interior columns below the ninth floor are reinforced with star-shaped structural steel verticals, in some columns helped out by straight rods and are founded on spread reinforced concrete footings. Above the ninth floor straight rod reinforcement alone is used. The typical column section is 20 × 20 ins., though in a portion of the first floor that is to be used for a store, aesthetic considerations compelled circular sections. Typical beams are 8 × 21 ins. in section and 16 ft. 11 ins. in span; typical girders are 25 ft. 9½ ins. in span and 12 × 24 ins. in section. Both are reinforced with loose twisted square rods. Floor slabs are 3½ ins. thick, reinforced with 5-16 in. square rods, 8 ins. c. to c., and ¼-in. square shrinkage rods, 2 ft. c. to c. at right angles to the main reinforcement. All floors were designed for live loads of 120 lbs. per sq. ft. The concrete in the foundations, girders, beams, and floor and roof slabs is a 1:2½:5 mixture; that in the columns is 1:2:4. Such a break in the quality of the mix between column and girder has since been prohibited by the Philadelphia building regulations, adopted after the erection of the Boyertown Building. These new rules require the same mixture in all structural parts of a building.

These general details are of statistical interest only. It is in the design of the side wall foundations that the Boyertown Building departs from standard reinforced-concrete design and adopts, in cantilever supporting girders, one of the well known details of steel construction. It has long been the practice, in crowded city blocks, where space is valuable, to carry the walls of new buildings on beams cantilevered back to columns inside the building, in order to avoid the dangerous and expensive underpinning of the adjoining foundations.

This solution was applied in the Boyertown Building. On either side of the narrow lot heavy stone and brick walls extended over the dividing property line and rather than to replace these walls by a new party wall, offset columns were placed inside the building and the entire upper works of columns and walls carried on cantilever beams to these lower columns. The floor plan of the basement and first floor given in Fig. 1, shows the position of these offset columns. Owing to certain differences in the nature of the adjoining walls the offsets on the two sides are of different magnitudes. In their general design the cantilevers on either side are similar, but the sizes given in the description and in Fig. 2 apply to the larger beam in which the offset is greater.

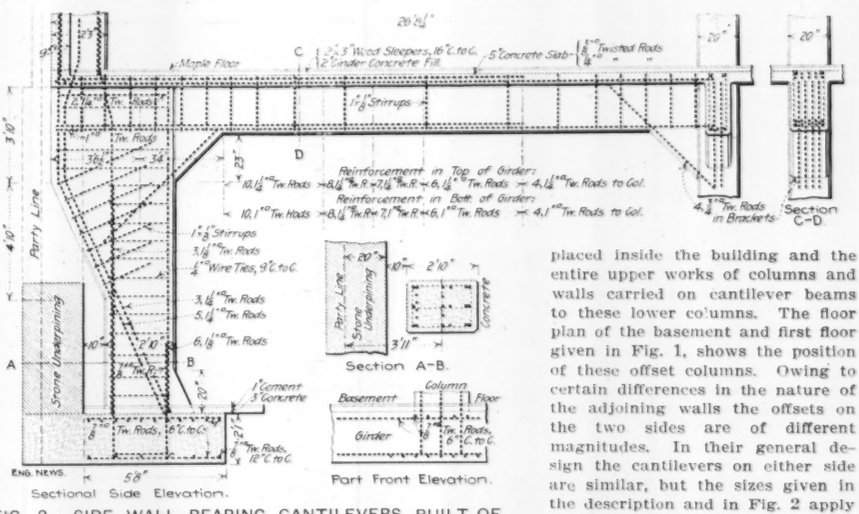


FIG. 2. SIDE WALL BEARING CANTILEVERS BUILT OF REINFORCED CONCRETE.

Along the entire length of the side wall a continuous foundation of reinforced concrete, 2 ft. 1 in. deep and about 6 ft. wide was built, extending out from under the adjoining stone foundation walls and heavily reinforced in the lower side against the eccentric loadings from the columns. Upon this foundation 26 × 34-in. columns were erected with centers 3 ft. 11 ins. inside the property line. From each of these columns, which were spaced at intervals of 8 ft. 6 ins. along the wall—or one-half the interior column spacing so as to reduce the load on the girders—a cantilever girder was extended, as shown in the figure, 3 ft. 6½ ins. from the center of the column. On the extreme end of this girder the columns for the upper ten stories were carried, the distance between the center lines of the upper and lower columns being 29 ins. The side walls consisted of brick laid up into 4-in. grooves molded in the sides of the columns.

In the rear of the building a much longer cantilever is used, although, as it carries a comparatively small load, it is not nearly so heavy as the side wall girders. The small street or alley running along the rear entrance to the building does not, in itself, give sufficient room for the passage of wagons in loading and delivering goods, so one of the rear rows of columns was omitted (Fig. 1), thus making a rear platform on level and connecting with the alley and covered by the second story floor slabs. The floor system on each floor above is carried at this point on a cantilever girder from the column next inside on the line of the one omitted. The design of the girder is shown in Fig. 3. It is 7 ft. 6 ins. long, reinforced in the upper flange with steel angles and in the lower with twisted rods. In the upper stories where the columns carry only round rods the angles in the upper flange of the girder are replaced by rods. The fascia beam which is carried on the end of the cantilever has its 42 ft. span divided into spans of 27 ft. and 15 ft. by the girder. Fig. 4 shows the first floor of the bay in the back of the building where the cantilever girder is used.

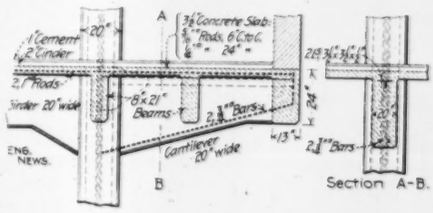


Fig. 3. Cantilever Beam Bearing Fascia Girder.

The cantilever principle is employed in several other connections on the building, but the two described above are the principle examples.

The top floor of the building is lighted, in addition to the regular side windows, by two rows of saw-tooth skylights, 17 ft. wide, in lengths of 20 ft. and 30 ft. respectively. The framework of these skylights is of reinforced concrete; the details are shown in Fig. 5. Through these skylights four of the main columns of the building are continued about 30 ft. above the roof to support a 25,000 gal. sprinkler tank for fire protection.

About one-third of the Boyertown Building is to be used for the storage and show rooms of the Boyertown Burial Casket Co. and the remainder for office purposes. The entire work of designing the architectural, structural and mechanical features was in charge of Messrs. Ballinger & Perrot, of Philadelphia, who furnished us the information from which this article was prepared.

LAND DRAINAGE AND RECLAMATION IN ITALY has been carried on to a very great extent, partly to improve the malarial conditions around the great marshes, and also to make these areas available for cultivation. Up to June, 1906, about 1,360,000 acres of land had been drained and improved, and put under cultivation. The cost of the work had been $36,265,640, paid by the government for the carrying out of the various enterprises or as subsidies to companies and private enterprises. At the same time works estimated to cost $2,077,730 were under construction, and the area yet to be improved is about 1,628,870 acres. This area includes the malarial districts around Rome. These were well drained by canals in the days of the Roman empire, but after its fall the canals and works were neglected. The cost of reclaiming the land is estimated at $40,000,000 and it is expected that the work will be undertaken by the government. The above information is taken from an article by an Italian correspondent of the engineering supplement of the London "Times."

CITY WASTES DISPOSAL AND STREET CLEANING.*

By PAUL HANSEN,† Assoc. M. Am. Soc. C. E.

The State Board of Health of Ohio has frequently been called upon by local health authorities for advice relative to methods of city wastes collection and disposal, but owing to lack of data on the subject it has been unable to render such assistance as has characterized its work in connection with water and sewage purification. With a view to greater usefulness along the lines indicated, the board has undertaken the study of the subject at first hand. The work so far done has been of a preliminary nature and has only involved visits to the larger cities for the purpose of obtaining descriptions of the methods developed locally, and the sending out of information blanks to all municipalities having over 5,000 inhabitants. During the coming year it is hoped to extend the work so as to include (1) tests on various garbage crematories in use in the state; (2) regular analyses of the various waste materials in a number of Ohio cities; and (3) a study of the progress being made in various cities in dealing with the problems involved. The information will be placed at the disposal of engineers interested in these problems.

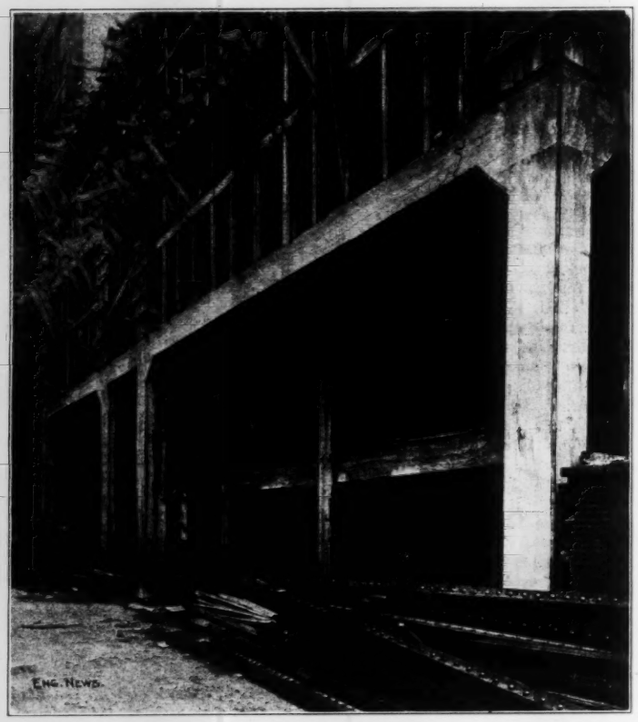


FIG. 4. VIEW OF FASCIA GIRDER SUPPORTED IN THE MIDDLE BY A CANTILEVER BEAM; BOYERTOWN BUILDING, PHILADELPHIA.

Methods of studying the disposal of solid wastes are in striking contrast to methods of studying the disposal of liquid wastes or sewage. In the latter case the character of the material itself is carefully ascertained by suitable analytical methods, and the knowledge thus obtained has proved of the greatest value in enabling engineers to design treatment plants with economy and precision. In the case of solid wastes, on the other hand, the only analytical data we possess in this country were made for a few of the Eastern cities. One of the important aims in connection with the contemplated extensive study of our subject is to somewhat relieve this dearth of analytical data by numerous analyses of various waste materials in a number of representative Ohio cities.

The class of waste which demands first attention, owing to its highly putrescible nature, is food waste, commonly designated as garbage. Garbage, while putrescible and frequently the cause of obnoxious nuisance, cannot be said to be responsible for the causation of specific disease, despite popular opinion to the contrary.

Methods of final disposal for garbage in use in this country at the present time are dumping on land, dumping in bodies of water, applying to land, as plowing in and burying, feeding to swine, cremation and reduction.

Dumping of garbage on land can scarcely fail of becoming a nuisance and renders the dump grounds unfit for habitation for many years to come. It may be conceivable that a locality can be found far removed from habitations, where the land is nearly valueless, and which might serve as a suitable dumping ground; but such a possibility is quite remote in the case of large cities, and even so, such methods of final disposal would be very uneconomical on account of long haul. It must be concluded that dumping is generally condemned. Dumping garbage at sea or in rivers and streams has also proved unsuccessful.

Application of garbage to land, if properly carried out, may be quite successful from a sanitary point of view. Plowing in is the generally preferred method, but to avoid nuisance the garbage should be covered with earth immediately after its delivery. Since garbage has considerable manurial value, it may be used to greatly improve certain soils and enhance the value of the land to which it is applied. Farmers generally object to garbage as a fertilizer because of the tins, crockery, and other matter which it contains. It, therefore, has in the raw state very little or no commercial value as fertilizer, and a municipality is fortunate if it can gain permission for using land without paying a bonus. The greatest objections to this method of disposal are economic, since, especially in large cities, it is difficult to secure the use of sufficient farm land near enough to avoid excessively long haul. In the case of the smaller cities, say not to exceed a population of 30,000, plowing in may under favorable conditions prove the most sanitary and most economical method available.*

Feeding to swine is a method of garbage disposal very extensively practiced, though in general the garbage used is somewhat selected. Hotel and restaurant garbage is preferred. There has been much adverse criticism of this method of disposing of garbage, usually on the ground that garbage contains poisons and germs that cause disease in the herd of swine. On the other hand, recognized sanitarians have upheld the practice when conducted under proper conditions of cleanliness; and as a matter of fact there is but little direct evidence of injurious results to the animals, and the objections to the practice would seem to be more theoretical than practical. In the speaker's experience, he has generally found piggeries where garbage is fed to be very foul and malodorous and a nuisance in the neighborhood. The contemplation of hog feeding with miscellaneous garbage must be rather repugnant to the average person, and the final word relative to its advisability from a sanitary standpoint cannot be spoken without more extended investigation. From an economic standpoint this method offers great advantages, since it is both profitable and applicable to a wide range of conditions.

The reduction of garbage to mineral ash by the application of intense heat appeals to most minds as the best and logical solution of the problem, and is strongly advocated by many as being the only ultimate satisfactory solution. Its merits from a sanitary point of view are certainly many, but the practical and economic difficulties are more than appear upon first thought. Among the latter are the high first cost and maintenance of suitably constructed furnaces, the difficulty of securing efficient operators, and the frequent difficulty of suppressing all nuisance. Nevertheless, cremation has great advantages, but these will not be fully brought out until more complete data are available relative to results obtainable, and until the installation of crematory plants is placed under the supervision of competent engineers.

Reduction of garbage is peculiarly an American development and is practically unknown abroad. It has for its object the extraction of grease and fertilizer. There are a number of types of apparatus in use. Their essential differences lie in the method of driving off moisture and extracting the grease; other differences lie in details of construction. The latest processes aim to retain all of the dissolved and suspended matters contained in the moisture of garbage, the same being quite rich in fertilizing value. The reduction process is represented to be sanitary, and in the sense that it disposes of garbage in such manner as to make it free from injurious germs and not subject to subsequent putrefaction, it fulfills its purpose. On the other hand, the process gives rise to a strong caramel odor, approximately described as the odor of sweetish burnt coffee, and which is very objectionable to many nostrils. The carrying power of this odor is also great. So far as the speaker's experience goes, he has never visited a reduction plant that was free from odors either on the inside or outside of the buildings. All this means that the reduction process is not free from objection, unless it can be located in a vicinity far removed from dwellings. In thickly populated districts this is not an easy matter and, aside from the fact that European garbage is low in grease, may account in some degree for its not being favored in the densely populated European countries. In the United States where reduction plants have been introduced they have nearly always been built as far removed from habitations as possible.

Until recently, reduction plants being in the hands of private companies, the actual costs of operation were closely guarded. In this state in the course of our investigation, it has been possible to get very complete figures for the municipally-owned plant at Cleveland and the privately owned plant at Dayton. It would appear from a study of these figures that as a general proposition reduction can be made at least to pay for itself with the average market prices for grease and tankage. In the case of Cleveland it is anticipated that sufficient profit will be made during the coming years to pay for a large part of the cost of collection also. It should be noted that the success of the reduction process depends on the treatment of large quantities of material, and its applicability is therefore limited to large cities, say with populations of over 100,000. It should be mentioned that many reduction plants have failed from causes that have not been generally known. Contributing causes were no doubt faulty design, improper operation, too great expectations with relation

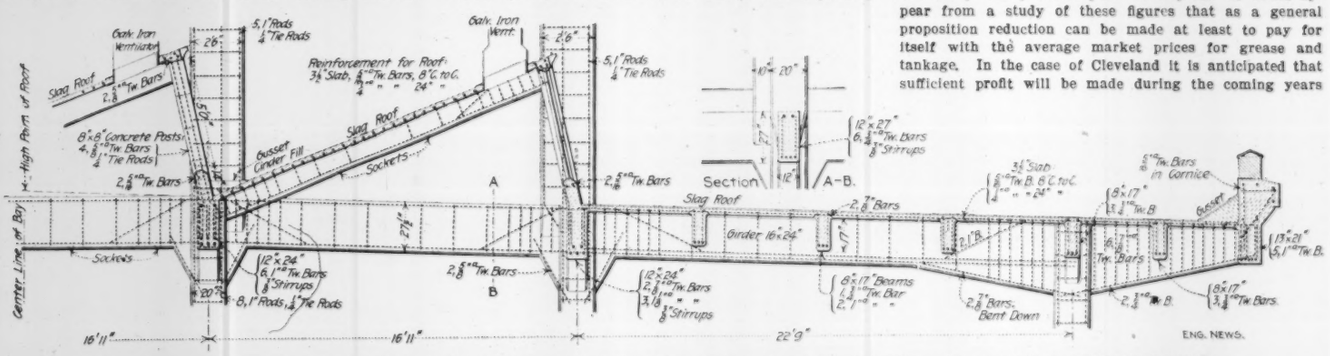


FIG. 5. REINFORCED-CONCRETE SAW-TOOTH ROOF CONSTRUCTION ON THE BOYERTOWN BUILDING, PHILADELPHIA.

*Abstract of a paper read before the Ohio Engineering Society, Feb. 12, 1908.

†Assistant Engineer, Ohio State Board of Health, Columbus, O.

[*See Eng. News, April 6, 1905, for editorial discussion of garbage disposal on land.—Ed.]

to profits, and over-capitalization of operating companies.

Most large cities in this country are being driven for reasons of both sanitation and economy to the adoption of either cremation or reduction for disposing of garbage. Granting that reduction plants can be located where odors will not give rise to complaint and that a crematory can be operated without creating disagreeable odors, the selection of the reduction or cremation system resolves itself into a problem of economy. This problem is not an easy one since its solution depends on a great variety of considerations too numerous to mention within the limits of this paper. Efforts have been made to secure by correspondence with cities throughout the country data which would indicate the relative merits of reduction and cremation from an economic point of view, but the data thus obtained were misleading owing to the impossibility of knowing all the local conditions.* One of these investigations, assisted in by the speaker in 1902, seemed to indicate that reduction was the more expensive to the taxpayer, but in the light of recent information the reverse would seem to be possible under municipal ownership and efficient management.

REPORT ON STREET CLEANING AND WASTE DISPOSAL, NEW YORK CITY.

A report on street cleaning and the disposal of the various classes of city wastes, that might almost be characterized as a treatise on the subject, although largely local in character, has recently been made public by the city of New York. The report is the result of investigations extending over a period of six months, made by a commission of engineers appointed by Mr. George B. McClellan, Mayor of New York City. The three members of this Commission are as follows: Messrs. H. de B. Parsons, Rudolph Hering and S. Whinery, M.'s Am. Soc. C. E., of New York City.

The investigations covered the five boroughs composing Greater New York. Besides looking into the operations of the various divisions of the Department of Street Cleaning, the Commissioners had chemical analyses made of garbage, ashes, rubbish and street sweepings, a mechanical analysis of rubbish, and determinations of the calorific value of garbage, ashes and street sweepings. Personal visits were made by the Commissioners to a number of American cities to study street-cleaning and waste-disposal methods in vogue there. Information was obtained from abroad by correspondence, and in addition, an arrangement was made with Mr. George A. Soper, M. Am. Soc. C. E., of New York City, to make a personal study of street cleaning in a number of European cities during a trip which he was about to make abroad.

Excluding snow, the total amount of refuse of all sorts collected by the city of New York and by private carts operating under permits in the year 1906, was 3,249,445 cart-loads, which is stated to be equivalent to 8,359,648 cu. yds., or 3,159,182 tons. The average per capita weight of refuse of various classes collected in New York City during the three years 1904-'06 was as follows: Garbage, 181 lbs.; ashes, 936; rubbish, 93; street sweepings, 260; total refuse, 1,470 lbs.

STREET CLEANING.

The general method of street cleaning in use in New York is described in the report as follows:

> The methods adopted in each borough for cleaning the streets differ somewhat according to the kind of pavement, location and character of the people. Generally speaking, the streets are swept by hand under a patrol system, each sweeper having a definite route to keep clean. The length of street assigned to each varies, of course, with the kind of pavement, the amount of travel and the other conditions which affect the area one man can clean. Each sweeper is furnished with a can carrier, five cans, one long-handled African bass broom with metal scraper attached to its back, one short-handled broom, one long-handled scraper (for asphalt or smooth surface pavements) and one metal dust-pan. Each sweeper is required to sweep over the entire surface of the pavement from one to three times daily, to collect the litter and sweepings into piles near the gutters, and to shovel it into the cans. These, when filled, are placed at certain points on his route ready for the collection cart. Should the cans not be emptied, he continues the work, collecting the material into piles at the gutters.
>
> Machine sweeping with two-horse machines is also employed to a comparatively small extent, the work being done at night.

The Commissioners are of the opinion that the general plan of organization of the New York Street Cleaning Department "is well devised and, with proper administration, should be efficient and satisfactory," but greater coordination and cooperation between the city departments is needed.

Of a total of 1,848 miles of different kinds of pavements in Greater New York at the beginning of 1907, 686 miles were macadam, 590 sheet asphalt, 230 granite, 74 block asphalt, 51 cobble, and the remainder miscellaneous. In Manhattan there were 248 miles of sheet asphalt out of a total of 433 miles of pavement, and 94 miles of granite.

The Commissioners' description of the various forms of street dirt is well worth attention:

> Street dirt is found upon the street under two distinct physical forms, though both have the same origin and are of the same general composition.
>
> Much the greater part of the refuse which reaches the surface of the pavements consists originally of coarse and often damp fragments, not readily taken up or blown about by ordinary breezes. In this primary condition street dirt can hardly be said to endanger the health of the populace, nor is it the source of much discomfort or damage. Even the fresh excrement of horses and germs of disease expectorated upon the street are not usually a source of infection while in a fresh and damp condition. If, however, the accumulated refuse be allowed to remain it will in time become finely pulverized by the action of the feet of horses and the wheels of vehicles. In this secondary condition it is street mud or slime in wet weather, and street dust in dry weather.
>
> A layer of slimy mud covering the streets acts as a lubricant between the pavement and the feet of horses, rendering the most desirable pavements—those with a continuous smooth surface—dangerous for the passage of horses, and greatly diminishes the loads that can be hauled. It pollutes the feet and clothing of human beings, and it is thus carried into residences and offices where, after it becomes dry, any disease germs it contains may find lodgment in the lungs and blood of the occupants.
>
> In the form of street dust it is a serious menace to health and a destructive and discomforting element of city life. Disturbed by street travel and carried about by even slight breezes, it is drawn with its disease germs into the lungs, and it permeates every crack and cranny of homes and business houses, damaging costly furnishings, clothing, and delicate goods, to say nothing of annoyance and discomfort.
>
> Notwithstanding these facts, the efforts of the street cleaning departments of nearly every city are principally directed to the removal of street dirt in its gross or primary form; or rather the methods of cleaning in common use remove only the coarser fragments, leaving the mud and dust upon the surface of the street.

The last paragraph of this quotation is noteworthy. In order to determine how much dirt is left on the streets by ordinary methods of cleaning, the Commissioners had some very interesting observations made. These consisted of careful cleanings and measurements of selected street areas after the ordinary cleaning had taken place. Placing the weight of dirt from sheet asphalt at 100, the comparative weights of dirt from these recleaned areas were 182 from block asphalt, 145 from one of the oldest wood block pavements in the city, and 912 from granite block.

Table I. gives various unit volumes and weights of street sweepings collected in Manhattan and Brooklyn boroughs in 1906.

TABLE I.—VOLUMES AND WEIGHTS OF STREET SWEEPINGS COLLECTED IN MANHATTAN AND BROOKLYN BOROUGHS IN 1906.
(Macadam excluded.)

	Manhattan. Volume, cu. yds.	Manhattan. Weight, tons	Brooklyn. Volume, cu. yds.	Brooklyn. Weight, tons.
Per mile of paved streets.	1,675	852	573	220
Per 1,000 sq. yds. pavem't	81	41	31	12
Per 1,000 population	325	165	218	84
Per 1,000 horses*	8,760	4,450	6,670	2,570

*The number of horses is obtained by assuming that the number given by the U. S. Census of 1900 has increased in the same ratio as the population. This gives 80,367 in Manhattan and 46,987 in Brooklyn.

One of the most valuable parts of the report deals with the efficiency and cost of street cleaning by machine sweeping, hand sweeping and street flushing by hand hose and by machinery. The well-known disadvantages of sweeping by the ordinary revolving brooms are stated: these brooms remove only the coarser fragments of dirt and are liable to be accompanied by much dust or much slime, according as to whether the sweeping is not or is preceded by sprinkling. The cost of machine sweeping, as estimated by the Commissioners, is given in Table II.

TABLE II.—ESTIMATED COST OF MACHINE SWEEPING.

Cost of one outfit (new):		
1 sweeping machine		$275.00
½ of 1 one-horse sprinkling wagon*		104.00
12 hand brooms, at 65 cts.		7.80
6 shovels, at 75 cts.		4.50
2 horses for sweeper		600.00
½-horse for sprinkler*		150.00
2½ sets of harness, at $25		62.50
		$1,203.80
Annual charges:		
Interest on this outfit, at 4%	$48.15	
Repairs & depreciation on tools, at 20%	90.76	
Depreciation on horses, at 15%	112.50	
Total annual charges	$251.41	
Or, for 310 days, per day		$0.81
Operating expenses per day:		
Maintenance of 2½ horses, at $1.35	$3.375	
Rent, storage of sweeper	.20	
Wages, 1 sweeper-driver	2.19	
Wages, ½ of sprinkler-driver	1.095	
Wages, 6 gutter sweepers, at $2.19	13.14	
15,000 gals. of water used for sprinkling, at $90 per 1,000,000	1.35	
		21.35
Grand total cost per day		$22.16

The above outfit will sweep, once, about 70,000 sq. yds. of street in one day of 8 hrs., and the cost per 1,000 yds. will be $0.317. The cost of loading the sweepings into carts and the cost of administration are omitted, because these items may be considered as costing about the same, whatever method of cleaning is employed.

*One sprinkling wagon required for two sweepers.

None of the pick-up street-sweeping machines "yet meets the conditions of every-day use."

Hand sweeping is considered to give better results than machine sweeping, provided the sweepers are "alert and reasonably industrious." Contrary to the usual belief, hand sweeping is not more costly than machine sweeping, as is shown by Table III. Hand sweeping is considered to be more effective than machine sweeping, especially where the pavements are rough.

TABLE III.—ESTIMATED COST OF HAND SWEEPING.

Cost of one outfit:		
1 hand cart		$10.00
5 cans for sweepings, at $2.50		12.50
4 hand brooms, at 65 cts.		2.60
1 shovel		.75
2 steel scrapers, at $2		4.00
		$29.85
Annual charges:		
Interest on outfit, at 4%	$1.19	
Repairs and depreciation, at 6%	17.91	
Total annual charges	$19.10	
Or, for 310 days, per day		$0.062
Cost of operation per day:		
1 sweeper		2.19
Total cost per day		$2.252

One such sweeper will clean satisfactorily 8,000 sq. yds. of pavement per day, and the cost per 1,000 sq. yds. will be $0.281.

Cleaning with water, or street flushing, may be done by either hose connected to fire or special hydrants, or by means of water under pressure in tank wagons, frequently called flushing machines. The ordinary hand flushing is wasteful of water, inasmuch as it is common practice to use a 1¼-in. nozzle discharging water under a hydrant pressure of 40 lbs. per sq. in., thus requiring about 235 gals. per min. to wash from 4,000 to 10,000 sq. yds. Assuming an average of 6,000 sq. yds. flushed per hr., the total cost per 1,000 sq. yds. with ordinary hand flushing would be $0.319; but by an improved method mentioned directly below this cost may be reduced to $0.251. In the improved method a special nozzle, equivalent in capacity to an ordinary 1-in. nozzle, throwing a fan-shaped jet having its longer axis parallel to the surface of the pavement, is used, and the pressure employed is not more than 30 lbs. per sq. in. With such a nozzle and 100 ft. of 2-in. hose, the discharge will not exceed 150 gals. per min., or 9,000 gals. per hr. Table IV. shows the estimated cost of hand washing by the improved means.

Street flushing machines "have been reported as rendering excellent service." They generally consist of a tank with two air-tight compartments connected by a passage above the water line. The water compartment has a capacity of 600 gals. Air under pressure is used in the other compartment and the design and operation is such as to give a water pressure when sprinkling of 35 lbs. per sq. in. A special nozzle dis-

*See publications of American Public Health Association; Vols. XXII., XXIII. Reports of the Committee on Disposal of Garbage and Refuse; Vol. XXX., Part I. [See Eng. News, Nov. 5 and Dec. 10, 1904, for editorial discussion and for correspondence on these investigations.—Ed.]

TABLE IV.—ESTIMATED COST OF HAND WASHING WITH IMPROVED NOZZLE.

Cost of one outfit:		
100 ft. of 2-in. hose, at 80 cts.		$80.00
One 1-in. nozzle, special		12.50
6 brooms, at 65 cts.		3.90
		$96.40
Annual charges:		
Interest on outfit, at 4%	$3.86	
Repairs and depreciation, at 150%	144.60	
Total annual charges	$148.46	
Or, for 310 days, per day		$0.479
Operating expenses per day:		
2 sweepers, at $2.19	$4.38	
57,600 gals. of water, at $90 per 1,000,000	5.184	
		9.564
Total cost per day		$10.043

If we assume that the work done would be at the rate of 5,000 sq. yds. per hr., or 40,000 sq. yds. per day, the cost per 1,000 sq. yds. would be $0.251.

charges a jet or a sheet of water on the pavement at a suitable angle so that by means of successive trips on a street the dirt is all washed to the gutter. The Commissioners state that the data on the cost of operating street flushing machines and the area cleaned per hour are divergent. They assume that one machine will clean about 3,500 sq. yds. of pavement per hr. and use water at the rate of 2 gals. per sq. yd. cleaned. Table V. shows the cost of street cleaning under these conditions.

The amount of dirt carried to the sewers under the street flushing plan may be reduced by care in operation and by using temporary dams in the gutters just above the sewer inlets. The report points out that the normal function of storm-water sewers is "to carry away with the surface water any detritus it may contain." With either kind of street washing, special hydrants may be employed in order to obviate the danger of interference with the fire service that is likely to result where fire hydrants are to be used by other than the fire department.

The costs of the various methods of street cleaning per 1,000 sq. yds. may be summarized as follows:

Machine sweeping	$0.317
Hand sweeping (patrol system)	0.281
Flushing by hose (as usually done)	0.319
Flushing by hose (suggested modification)	0.251
Flushing by machine	0.721

It is surprising to find (page 60 of the report) the statement that the moist or slimy street dirt resulting from sprinkling may in warm weather become "a prolific breeding place for disease germs." No exception can be taken, however, to the statement that this slime "defiles the feet and clothing of pedestrians" and is "a nuisance which is only tolerated because it is considered a lesser evil than the street dust resulting from imperfect street cleaning."

The average relative costs of cleaning different kinds of pavement equally well, assuming sheet asphalt as 100, are shown in Table VI., both in percentages and in dollars, the latter showing being on the basis of 100 cleanings per year per mile of pavement.

TABLE V.—ESTIMATED COST OF CLEANING BY FLUSHING MACHINES.

Cost of one outfit:		
1 flushing machine		$1,000.00
6 hand brooms, at 65 cts.		3.90
3 shovels, at 75 cts.		2.25
2 horses, at $300		600.00
2 sets of harness, at $25		50.00
		$1,656.15
Annual charges:		
Interest on outfit, at 4%	$66.25	
Repairs and depreciation on tools, at 14%	147.86	
Depreciation on horses, at 15%	90.00	
Total annual charges	$304.11	
Or, for 310 days, per day		$0.982
Operating expenses per day:		
1 driver	$2.19	
½-day of one helper	1.09	
Maintenance of two horses, at $1.35	2.70	
4 laborers collecting dirt in gutters, at $2	8.00	
Rent, storage of machine	.20	
Value of water used, 56,000 gals., at $90 per 1,000,000	5.04	
		19.220
Total cost per day		$20.202

Area of street cleaned in 8 hrs., 28,000 sq. yds.; cost per 1,000 sq. yds., $0.721.

In some cities the cost is somewhat reduced by using the same machine both night and day, two shifts of horses and men being employed.

As to the effect of the condition of the pavement upon the cost of cleaning, the consensus of opinion among 11 experienced New York district superintendents was that it cost 20% more to clean pavements in "fair" condition than those in "good" repair, and 40% more to clean those in "bad" condition. The Commissioners estimate that the total cost of cleaning the streets of the boroughs of Manhattan, the Bronx and Brooklyn ($2,245,000) might have been reduced by $374,000 if all the pavements had been in good repair.

Comparisons of costs of street cleaning in various cities are believed by the Commissioners to be of little value, owing to great differences in local conditions.

In making up estimates for the cost of street cleaning in the Borough of Manhattan, the Commissioners estimated that 10% of the streets, or 42 miles, would be flushed every day, 20%, or 84 miles, every second day, another like mileage every third day, 30%, or 126 miles, every seventh day, and 20%, or 84 miles, every tenth day. On this basis one-third of the total 420 miles of streets, or 140 miles, would be flushed daily. Assuming the use of water at the rate of 150 gals. per min., or, allowing for lost time, an average of 120 gals. per min. during eight hours of the day, this would require 4,250,000 gals. per day for the Borough of Manhattan. On the same basis 900,000 gals. would be required for the Bronx and 3,500,000 gals. for Brooklyn. In round numbers it is assumed that 10,000,000 gals. per day would suffice for the three boroughs. It is estimated that the total additional cost of street cleaning in Manhattan and the Bronx if flushing were used would be only $165,000 a year, and that this could be more than offset by enforcing the laws against street littering now in existence. Of course it should be remembered that the character of the street cleaning would be greatly improved. All water used should be credited to the Water Department and debited to the Street Cleaning Department.

The Commissioners are of the opinion that "if the street sweepers in New York worked industriously and were under proper discipline a great deal more work could be accomplished and the streets would be kept in a much cleaner condition."

The storage of street sweepings in bins underneath the sidewalk is recommended for the consideration of the Street Cleaning Department, in place of the use of the cans now employed by the hand sweepers.

Snow removal is considered by the Commissioners at some length, and a number of recommendations for handling this troublesome work are made. Most of these suggestions relate to departmental organization. The importance of clear and definite specifications for snow removal by contract is pointed out. Recommendations as to the payment for this work are as follows:

(11) That the payment to the contractor shall be based on a quantity of snow, ascertained by multiplying the area of the street, between the house lines, by 30% of the depth of snowfall in the storm considered. The records of the local offices of the U. S. Weather Bureau may be used to determine the depth of snowfall, or if found desirable, the Department of Street Cleaning may establish as many stations as may be necessary to determine the snowfall. As the areas of the street surfaces are now recorded on maps on file in the department, there ought to be no dispute as to quantities if the contract and specifications clearly define this method of measurement for payment.

(12) That the inspectors shall send to the department every 12 hours their reports of the work done during the last 12 hours, which reports shall clearly state the block or blocks that have been cleared by the contractor and the block or blocks in which snow has been piled but not carted away. The payment for such day's work shall be figured only for those whole blocks from which the snow has been carted away. Should the contractor fail at a subsequent time to remove the snow piles, then he shall receive only a portion of his contract price for such blocks.

(13) That if a second snowfall comes before the first has been all cleared away, the work on the areas already cleared or piled shall begin de novo, and shall be treated as an independent snowstorm. That the quantities from all blocks in which the work of clearing had not been commenced before the second fall shall be figured at 30% of the combined depth of the two snowfalls.

TABLE VI.—RELATIVE COSTS OF CLEANING VARIOUS CLASSES OF STREET PAVEMENTS, ASSUMING SHEET ASPHALT AS 100.

	Per cent.	Excess cost over sheet asphalt.
Sheet asphalt pavement	100	...
Wood block pavement, new	105	$26.40
Asphalt block pavement	115	79.20*
Brick pavement	120	105.60
Wood block pavement, old	125	132.00
Medina block pavement	130	158.40
Granite block pavement	140	211.20
Belgian block pavement	150	264.00
Cobblestone pavement	300	1,584.00

*Average condition.

It is suggested that it might possibly be advantageous if the snow-removal contract were made to cover a term of years, "so that the contractor could arrange for a permanent organization and equipment."

FINAL DISPOSITION OF REFUSE.

In the year 1906 the garbage of Greater New York, measured in cart-loads, was disposed of as follows: At sea, 124,883 cart-loads; at Barren Island, 229,986; incinerators, 18,206; inland dumps, 15,105 cart-loads. A total of 6,701 cart-loads of steam ashes was sold, and 80,319 cart-loads of other ashes were disposed of on "inland dumps." Of 2,631,163 cart-loads of mixed ashes and rubbish, 1,106,770 were used for filling at Riker's ~~Island~~ and 532,951 cart-loads were taken away by a company [presumably for filling]; the remainder was variously disposed of. Of paper and rubbish, 54,083 cart-loads were disposed of in incinerators and 10,444 cart-loads were dumped.

It might be thought that the street sweepings of New York could be readily disposed of for fertilizing purposes, but the Commissioners state that nearly 98% of the mass is inert material and that the "cost of distributing the street sweepings on farms is disproportionate to their value;" besides which, commercial fertilizers are not only cheaper but more convenient. It is becoming more and more difficult to dispose of stable manure, even, to farmers at a price that will cover the expense of delivery.

The continuation of garbage reduction is recommended for a large part of the city. On this point the report states:

Because of the immense quantities of garbage, which must be disposed of daily without serious interruptions, and because high temperature incineration of refuse containing garbage has not yet been introduced in any of the boroughs, we hesitate to recommend at the present time incineration of garbage for all of the boroughs.

We have given careful study to the process of reduction as now conducted at Barren Island, and we recommend that the garbage collections of the boroughs of Manhattan, the Bronx and Brooklyn be disposed of by reduction, by contract under suitable specifications. Provision should be made for a more prompt delivery to the reduction plant than at present, as fresh garbage is less offensive than stale garbage.

It is interesting to note in passing that the report cites decisions of the U. S. Supreme Court (199 U. S., 306, and 199 U. S., 325) to the effect that "household garbage is not private property which can be disposed of by the producers in a manner contrary to the requirements of city ordinances or the rules of a board of health."

For the boroughs of Queens and Richmond it is recommended that the refuse collected be burned in "carefully designed and constructed" furnaces at high temperatures. It is believed that if these destructors are properly located the heat can be utilized for power purposes, thus reducing the cost of destruction. The furnaces should be located at district receiving stations, thus shortening the hauls for the refuse. The ashes from the destructors should be taken to land fills.

For the boroughs of Manhattan, the Bronx and Brooklyn it is recommended that rubbish be burned at receiving stations and that the ashes from these furnaces be added to the ash and street sweeping collections, the combined material to be taken to land fills. Heat utilization could be practised at these rubbish incinerators, also.

The laboring force in the present Street Cleaning Department is characterized by the Commissioners as follows:

Whatever may have been the cause or causes, the fact is that the present laboring force is not accomplishing the work it should for the pay received, although no body of laborers in the country engaged in work requiring equal skill and physical exertion is more liberally paid. Many of them are listless, sluggish in their movements, and careless with their work. The impression given the observer is that their only interest in the work is to put in the required number of hours on the street with the least possible exertion. There is an absence of the spirit and enthusiasm which characterizes the work of the efficient laborer in any field. These remarks do not, of course, apply to all the men. There are many exceptions, but they do apply to the force as a whole.

Pensioning long-service employees and those disabled in the service is spoken of approvingly, but inasmuch as such a plan if introduced in one department would have to be adopted for all, the matter is merely recommended for consideration.

It is recommended that the Street Cleaning Commissioner be appointed permanently, subject to removal by the Mayor.

On the subject of cooperation between the various city departments, the report states:

It is an impossibility, in our judgment, to keep the city satisfactorily cleaned unless a full cooperation between the various city departments is established, so that they all work harmoniously with the same object in view—the improvement of the city, and its maintenance in a clean, tidy and sanitary condition. Cooperation requires readjustment in some of the detail practices on the part of each department and bureau, in order that the results of their joint effort will prevent duplication of work and promote the affairs of the municipality rather than those of the individual departments.

With the exception of the departments or bureaus under the Borough Presidents, all the branches of the municipal government are under the Mayor, who appoints the executive heads and has the power of removal. We have failed to find any practical cooperation on the part of others to assist the work of street cleaning and city scavenging. There are plenty of regulations and ordinances, which, if enforced, would effect a great difference in the condition and appearance of the city.

There must be cooperation if satisfactory results are to be obtained. In our opinion, a united effort on the part of all will accomplish marked results, but if the division of responsibility is such as not to make it attainable, then changes by law should be made. Lack of cooperation between departments results in the useless expenditure of the city's money.

We append, in Appendix D, an opinion of former Corporation Counsel George L. Rives, which mentions some of the assistance that the Department of Street Cleaning should receive from the other departments.

Importance is given to the desirability of improving the accounting and cost-keeping methods of the Department, as follows:

Facilities were given us to get necessary information from the books and records of the department, and if we were not able, in some instances, to get as complete and accurate data as was desired, it was the fault of the system of accounting and recording used, rather than of the department personnel.

While we did not make an examination of the method of bookkeeping practiced in the department, we recommend that what is known as uniform municipal accounting, already in use in a number of other cities, should be adopted by the Department of Street Cleaning.

It is obvious that unless costs and other results are reduced to the same standards and denominations, intelligent comparison between them and similar accounts in other cities is impossible. This is the main reason why accurate and useful data on street cleaning and waste disposal are now so difficult to obtain in a shape for intelligent use.

The accounts and records of the department should be kept in such completeness and detail that the quantities and costs, not only of the whole work, but of all the subdivisions, can be readily and accurately ascertained. Unless this be the case, the bureau heads will not be able to keep in sufficiently close touch with the details of the work to detect causes of waste or of undue expense, and to exercise that prompt and full control over the operations of the department necessary to secure economical results.

We also recommend that all the yearly reports of the department be made in a standard form, so as to clearly show the year's progress in a way that it can be compared with that of former years. If these reports are to be of any value, they should be of such form and completeness that any one can understand them and form an intelligent opinion of the efficiency and economy with which the department business has been handled.

With these objects in view, we recommend that the bookkeeping of the department be remodelled in accordance with the latest and most improved system of municipal accounting, and that, if necessary, special statisticians be employed to collate exact data relating to quantities, costs and other useful details of the department's work. In this way all valuable information will be recorded in a useful form, and the cost and efficiency of all the several kinds of work will be available for future guidance.

STREET CLEANING IN EUROPE.

In his report on "Street Cleaning in Foreign Cities," Mr. George A. Soper states that he visited about twenty European cities of 300,000 population and over. Little attempt at standardizing methods as between cities was observed, owing to the fact that European, like American, cities have generally developed their street-cleaning methods independently. Particular mention is made of the free and effective use of street flushing by means of hose in the city of London; of the very efficient manner in which streets are sanded to prevent slipperiness in Hamburg; of the cleaning of sidewalks and flushing of gutters with water in Paris; and of the particularly skilful and discriminating way in which street sprinkling is carried out in Westminster, "amid the densely congested and rapidly moving street traffic."

Engineers for the heads of street cleaning departments were usually found in the larger cities. It was also found that the street cleaning department was often "a branch of a larger department which has charge of the construction and repair of all structures between the house lines." This centralized responsibility for everything pertaining to the streets is recommended to the consideration of American cities.

Uniforms for the workmen of the street cleaning departments were found to be customary in the European cities, "but these were invariably quiet and inconspicuous, as compared with those in New York, which were purposely designed by their inventor, Col. Waring, to be as conspicuous as possible."

In the cities with the cleanest streets, the cleaning work is generally done in two divisions, day and night. The coarser materials are removed in the daytime and the more careful cleansing is done at night. The best results as to cleanliness are obtained with hand hose and squeegees, but this is the slowest and most expensive of the methods used. Flushing gutters is practised generally in European cities of all sizes, but not so extensively elsewhere as in Paris. The nozzle used for hose flushing "was generally circular in form, and about ¾-in. in diameter," and "was often provided with a deflector, by which the shape and force of the stream could be regulated." The hose was often 1½ ins. in diameter and ranged in length from 50 to 200 ft. In the city of London the nozzle discharge is at the average rate of about 96 U. S. gals. per min. Nozzle men receive extra pay.

Sprinkling carts are used to moisten and loosen the street dirt, and the hose to sweep it away. No automatic flushing machines were seen.

In some places, and notably in Charlottenburg (near Berlin), street sweepings are stored for six months or so and then used to fertilize flower beds in city parks.

Nearly all refuse in foreign cities is mixed before collection. Household refuse generally contains less kitchen waste, less paper, and apparently less unburned coal, than in America. Charlottenburg was the only city in which house refuse is separated before collection in a manner at all comparable with the New York practice.

The extent to which house refuse is used on land is noted by Mr. Soper as follows:

It was surprising to note how much household refuse was put upon land in one way or another. Berlin disposes of all of its household refuse in this way, as do Amsterdam, the city of London and many other large and small places. Until very recently, the city of Paris sold all its decomposable house refuse as manure for the cultivation of sugar beets. The city of Belfast burns only about one-tenth of its municipal refuse, the rest going upon land. Salford burns about half. Manchester owns 3,700 acres of rural estates purchased for the sole object of receiving city refuse. Refuse from the city of Glasgow is sold to farmers scattered in 15 counties in Scotland. All of the cities here mentioned, except Berlin, have refuse destructors also.

The sanitary advantages of the common practice of burning refuse in Great Britain are recognized by Mr. Soper, but he states that less is known concerning the cost of this method of disposal than is known of the cost of utilizing refuse on land. He adds:

When the total cost of burning is considered, the process often appears less economical than is popularly supposed. In fact, counting the cost of repairs, interest on the investment, sinking fund charges and the wages of the attendants, it not infrequently happens that it would be cheaper for a foreign city to turn the household refuse into manure and apply it to land.

To partly offset the cost of burning refuse, efforts are usually made to utilize the heat produced to raise steam for producing electric light, to pump sewage or water, to operate machinery and for many other purposes. The residue is used for many purposes, such as the making of concrete, mortar, bricks and asphalt pavements.

In concluding his report, Mr. Soper lays particular emphasis on the need of centralizing the responsibility for the repair and cleaning of pavements; of putting a competent person at the head of the Street Cleaning Department, "preferably an engineer experienced in sanitary work;" and he commends a form of organization "somewhat military in character" but not "carried beyond the point required to fix responsibility and insure a proper execution of orders."

The importance of sprinkling congested streets with sand and gravel when they become slippery is mentioned.

In conclusion we may note that the report of the Commission contains a large mass of tabular information on street cleaning and refuse collection and disposal in New York City and also in a number of foreign cities.

THE ABOLITION OF THE STATE BOARD OF Health and the State Sewerage Commission of New Jersey and the creation of a new State Board of Health, was provided for by a bill passed by the legislature, which adjourned on April 11.

A REPORT ON THE POLLUTION OF THE DELAware River within the State of New York has been made by the Engineering Division of the New York State Department of Health and is published in the Department Bulletin for March, 1908. The population on the 2,650 sq. mi. of drainage area is sparse, averaging only 29.5 per square mile for the total population and 15.06 for the rural population alone. There are no cities in the drainage area and the largest village (Port Jervis) had a population of only 9,685 in 1905, which was more than one-third of the total population of the 15 incorporated villages in the drainage area. Of the incorporated villages, only Port Jervis, Liberty and Stamford have public sewerage systems, and the sewage of both Liberty and Stamford is treated before it is discharged into the river. The chief manufacturing industries which discharge their sewage into the river are 23 acid factories, 74 creameries and 2 tanneries. There are no public water supplies drawn from the Delaware River within the State of New York nor from its tributary streams, except at the headwaters of the latter. The nearest Pennsylvania or New Jersey municipality taking its supply from the Delaware after the latter leaves the State of New York is Belvidere, N. J., about 53 miles below the New York line. As a result of the investigation it is recommended that in order to maintain the "present relative purity" of the river within the State and in order that all pollution may ultimately be eliminated, the general policy of the Department be:

(a) To prohibit the discharge of sewage from any new sewerage system constructed in the future, unless these are provided with proper and adequate works for sewage purification.

(b) To prohibit any extensions or additions to existing sewer systems unless disposal works are provided for a purification of the sewage of the entire system.

(c) To require under the Public Health Law, or otherwise induce, the villages which now discharge raw sewage into these waters, to introduce means for sewage purification as rapidly as it is possible and practicable.

It is also recommended that in the future the Department deny all applications to discharge into the river industrial wastes from new industries or to increase the discharge of wastes from existing industries, unless these wastes are suitably treated. It is stated that the conclusions and recommendations are in accordance with resolutions adopted at the joint meeting of the State health departments of New York, New Jersey and Pennsylvania, held at Atlantic City, N. J., on Sept. 14, 1906. On the date just named it was agreed by the representatives of the three States that the question of the pollution of the Delaware and its tributaries would be investigated in behalf of each State, and that active steps would be taken to remove existing pollution and to prevent additional pollution in the future.

THE DESIGN OF BUTTRESSED DAMS OF REINFORCED CONCRETE.

By R. C. BEARDSLEY.*

Within the last decade the buttressed type of dam has taken rank with the solid vertical-faced dam, and bids fair to become the standard construction of the future. When the Croton dam was under design, the engineers considered this type, and reported that, were it not for lack of precedents and popular bias, it would be the best type to build.

Mr. Edwin Duryea, in the discussion on the paper "Lake Cheesman Dam and Reservoir," read before the American Society of Civil Engineers on May 4, 1904, makes the following statement, referring to the design for a dam 126 ft. high:

> While the buttress design proposed for this dam lacks conservatism, it being higher than any buttress dam yet built, no other valid objection can be made to it, except the possibility (a very remote one even in California) of its injury by earthquakes.

It will be but a very few years now till precedents will be plentiful. Last year the writer designed a reinforced-concrete buttressed dam 50 ft. high which was built at Sulphur Springs, N. C., and the Ambursen company has recently finished a dam 60 ft. high.† Dams of this height will be rapidly succeeded by much higher structures, as the security of design and the permanence of the materials is demonstrated by the cases cited as well as though they were of the highest.

If there is any one feature of this type more pronounced than any other, it is that every factor of uncertainty, other than the crushing strength of the concrete, is practically eliminated. The engineer has only to compute the strength of beams and columns, while in designing a solid dam he must calculate the resistances to overturning and to sliding, the cantilever effect, the floating effect, the influence of ice expansion, temperature effects on large masses of concrete or masonry, etc., etc. The buttressed dam owes its simplicity to the fact that its upstream face is made so flat in slope that the water pressure has a vertical component sufficiently in excess of the horizontal to hold the structure solidly down upon the river bottom against the forces acting to overturn it or to slide it bodily downstream.

In designing a dam of the buttressed type the dead-load pressure upon the foundation should be determined first. The first step is to find the center of gravity of the weight areas. To do this the buttresses and deck have to be considered separately.

First, take the buttress. (See Fig. 1.) In the example used, this has a batter of 12 ins. in 50 ft., the base being 36 ins. thick the full length up and down stream. To get the center of gravity of the buttress $A'\ B\ C'$, use the formulas:

$$z = \frac{L'}{4}\,\frac{3d+b}{2d+b}$$

$$y = \frac{H'}{2}\,\frac{d+b}{2d+b}$$

As a check on the calculation, the center of gravity must be on the line drawn from C' to the middle of the base.

To get the cubic contents of the buttress $A'\ B\ C'$, find the average thickness from the formula

$$t = \frac{1}{3}\,(2d+b).$$

*Hydraulic Engineer, Roberts & Abbott Co., Cleveland, Ohio.

†Ellsworth, Me., 64 ft. high, Eng. News, May 23, 1907.

In this example the area of $A'\ B\ C' = 1{,}104$ sq. ft., and $t = 2\frac{1}{3}$ ft. Taking w, the weight of 1 cubic foot of concrete, as 145 lbs., we have for the weight of the buttress per lineal ft. of dam

$$\frac{1{,}104 \times 2\frac{1}{3} \times 145}{13} = 28{,}733 \text{ lbs., acting at point } G.$$

The deck of the dam is a trapezoid, the center of gravity of which is found by the well-known methods to be at G' and the weight per foot of dam is 19,865 lbs. The resultant of the two weights G and G' is found to act at G''.

To determine the effect of the water vertically

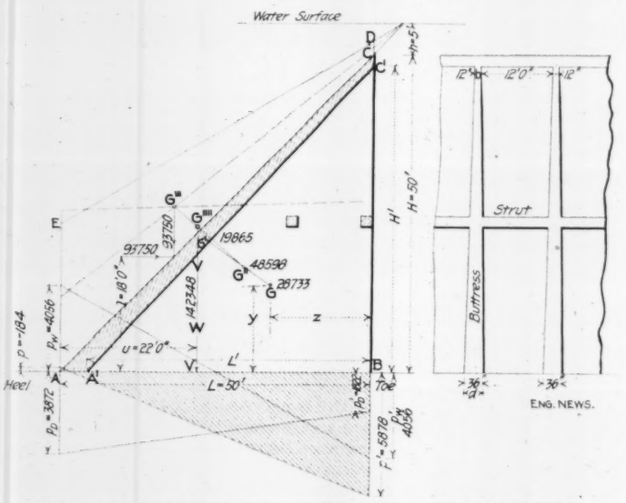


FIG. 1. BUTTRESSED DAM, 50 FT. HIGH, WITHOUT APRON.

above the dam, construct the trapezoid of pressure $A\ E\ D\ C$ such that

$$EA = \frac{(H+h)\ 62.5}{w} = \frac{(50+5)\ 62.5}{145} = 23.7 \text{ ft.}$$

and

$$DC = \frac{h \times 62.5}{145} = 2.15 \text{ ft.}$$

These distances are laid off to the same scale as used in drawing the dam.

The center of gravity of this trapezoid is found to be at G'''. The weight of the water vertically over the dam and carried by the dam is equal to the area of the trapezoid multiplied by 145, or 93,750 lbs.

The resultant center of gravity of G'' and G''' is at G''''. This is the point where the sum of all the weights, namely 93,750 + 19,865 + 28,733, or 142,348 lbs., is applied. A vertical dropped to the base of the dam from G'''' cuts the base at V, a point whose distance from A will be denoted by the letter u.

For calculating the pressures on the foundation, two conditions are considered: (1) when $u = L/3$; (2) when u exceeds $L/3$.

The third possible condition, u less than $L/3$, never occurs in a dam of the buttress type. The deck is always built at an angle not exceeding 45°, and when an apron on the downstream side is provided it is not given a flatter slope than that shown in Fig. 2, where $u = L/3$. As the deck is made flatter and the apron steeper, the center of gravity moves toward the center of the base, and therefore in practice u is never less than $L/3$.

The formulas to be used are:

$$\text{When } u = \frac{L}{3},\quad p_d = \frac{2W}{L},$$

$$\text{When } u > \frac{L}{3},\quad p_d = \frac{2W}{L}\left(2 - \frac{3u}{L}\right)$$

The value p_d is the pressure at that edge of the foundation which is nearest the line of action of the resultant W. The distance u is to be measured from the same edge. The pressure at the opposite edge, p'_d, is in any case given by the formula

$$p'_d = \frac{2W}{L} - p_d.$$

In the case of the first formula the diagram of pressures is a triangle, and $p'_d = 0$, which result also follows from the formula.

Ordinarily, p_d will be the pressure at the heel or upstream edge, as shown in Fig. 1.

In this example, Fig. 1, $u = 22$ ft. and therefore exceeds $\frac{L}{3}$. Calculating the pressures by the above formula (second formula), we have

$$p_d = \frac{2W}{L}\left(2 - \frac{3u}{L}\right) = \frac{2 \times 142{,}348}{50}\left(2 - \frac{3 \times 22}{50}\right) = 3{,}872 \text{ lbs. per sq. ft.}$$

$$p'_d = \frac{2W}{L} - p_d = 1{,}822 \text{ lbs. per sq. ft.}$$

Having the pressures due to all vertical forces, we must now find those due to the horizontal water pressure. Under the action of this force, the dam acts as a cantilever. The distance of the point of application of the water pressure P above the base, l in Fig. 3, is found as shown there, or as in Fig. 1. Then we have the general formula for beams, $M = S\ R$, where M is the bending moment, S the pressure at the extreme fiber

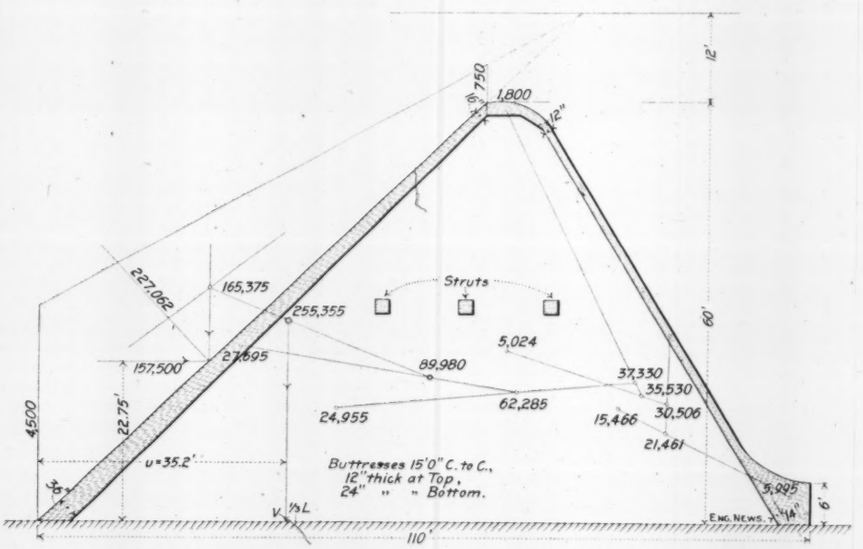


FIG. 2. 60-FT. BUTTRESSED DAM WITH APRON; MAXIMUM DEPTH OF WATER OVER CREST 12 FT.

and R the section modulus. Here $M = P\,l$, $S = \frac{b\,L^2}{6}$, and $R = \frac{L^2}{6}$, where b is the width of the section under consideration, here taken as one foot, so that (all dimensions being in feet),

$$R = \frac{L^2}{6}; \text{ also } P = \frac{(H+2h)}{2}\,62.5\,H.$$

In the example of Fig. 1, where the slope of the deck = 45°, the vertical and horizontal components of the water pressure are equal so that

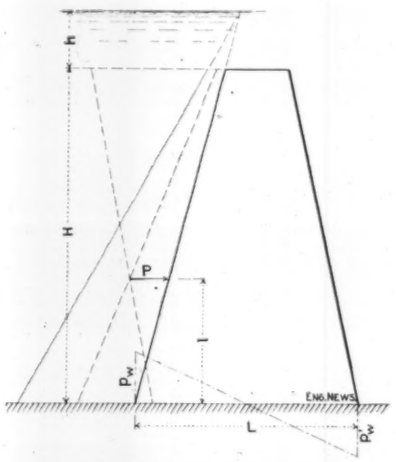


Fig. 3. Action of Water Pressure.

$P = 93{,}750$ lbs.

Then $M = 93{,}750 \times 18 = 1{,}687{,}500$ lb.-ft. The section modulus of the base is

$$R = \frac{50^2}{6} = 417.$$

Therefore,

$$p'_w = p_w = \frac{1{,}687{,}500}{417} = 4{,}056.$$

The resultant pressures due to both vertical loads and horizontal thrust are:

$$p' = p'_d + p'_w = 5{,}878 \text{ lbs. per sq. ft.}$$

and $$p = p_d - p_w = -184 \text{ lbs. per sq. ft.}$$

These pressures are per foot of length of dam. However, as they are actually concentrated on the area covered by the buttresses, which are 3 ft. wide and 13 ft. on centers, the pressures at the base of the buttresses will be larger in the ratio of 13 to 3. That is:

$$p' = \frac{5{,}878 \times 13}{3} = 25{,}476 \text{ lbs. per sq. ft.,}$$

and $$p = \frac{-184 \times 13}{3} = -797 \text{ lbs. per sq. ft.}$$

The pressure of 25,476 lbs. per sq. ft. would be considered quite safe by most engineers. As a general rule, 30,000 lbs. should not be exceeded unless the concrete is unusually well reinforced and 20,000 would represent the minimum stress allowed. If the footings are set in trenches cut in solid rock, higher pressures may be used than would be proper where they merely rest upon the natural rock surface.

The negative pressure, or tension, of 797 lbs. per sq. ft. at the upstream edge may be neglected, as a seepage trench is constructed along this edge of the dam which is so thoroughly anchored to the rock along its whole length that this lifting tendency is more than counteracted.*

The design of a dam with apron is carried out in the same way, the calculations being only slightly more complicated by the additional sections, the center of gravity of which must be found. Thus, Fig. 2 shows a dam 60 ft. high above tail-water and subjected to a head of water of 12 ft. over the crest. The buttresses are 15 ft. on centers, 12 ins. thick at the top and 24 ins. thick at the base.

Locating the center of gravity of the various sections of concrete and the vertical load of water, we find that $W = 255{,}355$ lbs. per foot of dam. The vertical passing through the center of gravity of the systems cuts the base practically at the upstream edge of the middle third, so that $u = \frac{L}{3}$.

(If the weight of water passing down the apron were considered, u would be greater than $\frac{L}{3}$.)

Then

$$\frac{2\,W}{L} = p_d = 4{,}643 \text{ lbs. per sq. ft.}$$

is the pressure at the upstream edge (average per lineal foot of dam). The moment of the water pressure is

$$M = P\,l = 3{,}582{,}125 = \frac{S \times 110^2}{6},$$

whence $p_w = S = 1{,}777$ lbs. per sq. ft. per lineal foot of dam. Then, as p_w is negative at the upstream edge

$$p = 4{,}643 - 1{,}777 = 2{,}866 \text{ lbs.}$$

The buttress being 2 ft. thick at the base, the actual pressure per square foot at the upstream edge is $\frac{2{,}866 \times 15}{2} = 21{,}495$ lbs. per sq. ft.

The pressure at the downstream edge is $p'_w = 1{,}777$ lbs. per sq. ft., and as $u = \frac{L}{3}$, the diagram of stress is a triangle and p'_d is zero. Therefore, $p' = 1{,}777$. The actual pressure per square foot at the downstream edge is $\frac{1{,}777 \times 15}{2} = 13{,}328$ lbs. per sq. ft.

These examples show that the pressures are apt to be much larger in the case of the gravity dam without an apron than in the case where there is one, owing to the wider base made by the apron.

When the dam exceeds about 30 ft. in height, there will have to be struts provided to prevent the tall, thin buttresses from buckling. An empirical rule is to place these struts at vertical distances of 24 times the thickness of the buttress, and horizontally about 10 thicknesses apart. The struts should be reinforced the same as columns, but with enough additional steel on the under side for beam action to carry the dead weight of the concrete.

Ice is a deadly foe to all dams, and especially is this true of those dams built of inflexible materials, which cannot give a few inches under the pressure. Several recently built reinforced-concrete dams of the buttress type were badly damaged last winter by ice expansion. While in rare instances the river may be so narrow that the shores take the thrust, usually the dam creates a large reservoir and makes an ideal ice field.

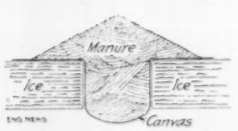


Fig. 4. Method of Protection Against Ice Thrust Used at Mazeppa, Minn.

Various authorities place the shoving force of a field of ice at 12 to 25 tons per sq. ft. The modulus of elasticity of ice is about 180,000 to 360,000 lbs. per sq. in., and the maximum expansion due to temperature .00025 to .00066. Suppose, then, a field of ice 12 ins. thick forms on a pond. The compression, equal to the temperature expansion, being .00025 to .00066 per degree temperature rise, the pressure will be .00025 × 180,000 to .00066 × 180,000, or 45 to 120 lbs. per sq. in. If the higher modulus is taken, the pressure will be 90 to 240 lbs. per sq. in. It will be noted that the length of the sheet of ice has nothing to do with the pressure, but has much to do with the distance through which the pressure will act.

Suppose that in the case of the 60-ft. dam the ice shoves the crest with a pressure of 150 lbs. per sq. in., and that the ice forms 12 ins. thick. Then $M = 150 \times 144 \times 60 = 1{,}296{,}000$ ft.-lbs. $= S\,\frac{110^2}{6}$. We find that the foundation pressure produced is equal to $p = S = 642$ lbs. per sq. ft. average, or $\frac{642 \times 15}{2} = 4{,}815$ lbs. per sq. ft. actual.

In the case of the dam shown by Fig. 2, the total pressure at the toe thus becomes 4,815 + 13,328 = 18,143 lbs. per sq. ft.

The writer knows of but three practical ways of preventing damage due to ice expansion.

At Waldron, Ill., a new dam was built, about 50 ft. below an old one, the old dam not being disturbed. Much trouble had been experienced on the old dam from ice thrust, but since the building of the new dam, the water passing over the old dam has set up such a current that no ice could form between them. This indicates a practical way to prevent ice forming on the dam.

Another way is to keep a channel about 18 ins. wide cut clear across the river and just above the dam. This channel may be filled with manure and covered with a canvas to prevent freezing. See Fig. 4. This method was adopted at Mazeppa, Minn., where 36 ins. of ice is common, and it worked perfectly.

The third method, and the best, is to provide the dam with a movable crest so that at any time the operator at the power-house may raise and lower it, thus breaking up the ice.

THE NEW SHOPS OF THE INTERCOLONIAL RY. of Canada at Moncton, N. B., are reported well under way. The buildings at present under contract and partly completed comprise the following. Machine shop, 172 × 410 ft.; smith shop, 75 × 375 ft.; engine erecting shop, 80 × 375 ft.; boiler erecting shop, 50 × 375 ft.; boiler and tender shop, 50 × 375 ft.; riveting tower in boiler erecting shop, 25 ft. wide, 50 ft. long, 75 ft. high; freight-car repair shop, 135 × 365 ft.; planing mill, 80 × 200 ft.; dry house, 20 × 73 ft.; passenger-car repair shop, 100 × 365 ft.; passenger-car repair shop, 100 × 365 ft.; office and storehouse, 50 × 350 ft.; power-house, 70 × 200 ft.; gas producer, 50 × 50 ft. The machine, smith, engine, boiler, freight-car shops, planing mill and power-house have structural steel frames and concrete walls and roofs. The other buildings are to be constructed of reinforced concrete. Locomotive houses, turntables, stand-pipes, coaling plant, cinder pits and hoists, water columns, etc., will be added in the next few years, and those now in the old yard will be abandoned. The old shops and grounds will be leased for a steel and wood car manufacturing industry. The designs of the new shops were prepared by Mr. W. A. Bowden, Designing Engineer of the Dept. of Railways and Canals, under the direction of Mr. M. J. Butler, Deputy Minister and Chief Engineer of the Dept. of Railways and Canals. Mr. E. A. Wallberg is the contractor and Mr. W. B. MacKenzie, Chief Engineer of the Intercolonial Ry., is Construction Engineer.

REINFORCED-CONCRETE BARGES and pontoons of considerable size have been in use for some time in Italy and have proved so satisfactory that the Italian government has contracted for several large barges for use in harbor work. The first of these boats is of sufficient size and has been in use long enough to furnish a test of their utility. It is a double pontoon, built in 1897, 67 ft. long and 27 ft. out-to-out of the two parts, on which is built a boat-house for the Rome Rowing Club. In 1905 the Italian Navy Department built the "Liguria," a reinforced-concrete barge of 150 tons burden, 57 ft. long and of 18 ft. beam. This was first put upon the harbor work near Rome but has since been towed to and from many of the ports of Italy. The "Liguria" was so successful that the government built another 100-ton barge, on the model of which four others are now under contract. In several other places in the kingdom, notably across the River Po near Pavia, pontoons of reinforced concrete are used to carry small or light bridges. Most of these constructions have been carried on by Messrs. Gabellini of Rome. Several interesting photographs of the boats are shown in the March, 1908, number of the London "Concrete and Constructional Engineering."

*But where the base cannot take tension, or may not safely be assumed to have tensile strength, the author's method is inapplicable in all cases where it shows a resultant tension at one edge. In these cases it gives too small a value for the compression at the toe. The more usual methods of calculation must then be used. —Ed.

A TEST OF THE SCREW PUMP FOR FLUSHING THE KINNICKINNIC RIVER AT MILWAUKEE, WIS.

The tunnel and pumping plant for flushing the channel of the Kinnickinnic River at Milwaukee, Wis., were described and illustrated in our issue of Aug. 1, 1907, and we have recently received from Mr. Charles J. Poetsch, M. Am. Soc. C. E., City Engineer, a copy of the report on the official test of the pumping plant. This plant was built by the Allis-Chalmers Co., of

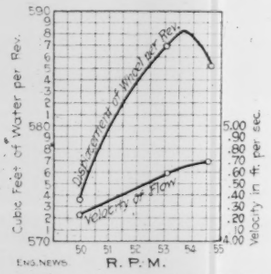


Diagram of Results of Tests of River Flushing Plant at Milwaukee, Wis.

Milwaukee, under a contract dated Feb. 11, 1904, and consists of a vertical, tandem-compound condensing engine, with cylinders 18 × 42 ins. and 38 × 42 ins. The shaft is in the line of the axis of the tunnel. It is connected to a screw 12½ ft. diameter, having a hub 7½ ft. diameter and six blades with a pitch of 9½ ft. The thrust is taken by a thrust bearing with multiple collars, as in marine engines. Steam is supplied by one of two horizontal tubular boilers 6 × 18 ft., with Hawley down-draft furnaces and Foster superheaters. The superheaters were guaranteed to add 100° of superheat to 5,000 lbs. of steam per hour, at 140 lbs. pressure.

The specifications required the engine to have sufficient capacity to raise and deliver 30,000 cu. ft. 3½ ft. high per minute, with a steam pressure of not over 140 lbs. at the throttle. Under these conditions it was to perform a duty of not less than 95,000,000 lbs. of water raised 1 ft. high for every 1,000 lbs. of steam used, no deductions to be made for steam containing 2% or less of moisture. The amount of water delivered was to be determined by the cross-sectional area of the tunnel in square feet, multiplied by the average velocity of water in feet per second. The head was to be determined by gages set in the pump well behind and a manhole in front of the pump wheel. No deduction was to be made for loss between these two points.

The velocity of water through the tunnel was to be determined before the official duty test of the engine in the following manner: (1) The length of the tunnel to be carefully and accurately measured. (2) With the engine running at the required speed, coloring matter was to be injected into the center of the tunnel at the intake end and the duration of its travel through the tunnel carefully timed.

The tunnel was pumped out for measurement as to length, and was carefully measured on May 28, 1907. The distance between the manholes at the lake inlet and in front of the pumping station is 7,176.75 ft. This length was used in calculating the velocity, as between the second manhole and the river outlet, the cross section of the tunnel changes and there are numerous obstructions, including the pump wheel, deflectors, etc. The tests for the velocity of water through the tunnel, which were also tests of the capacity of the pumping machinery, were made on Jan. 15, 16 and 17. A dark-red aniline dye was selected as the coloring matter, and saturated solutions were placed in the receptacle of a modified syringe inserted in the center line of the tunnel at each manhole. These were discharged at intervals, as directed by the observers; other observers noted the time of the appearance of the coloring matter in the river.

TABLE I.—TIME OF TRAVEL OF COLORING MATTER IN WATER.

From lake to river: Speed of engine, r. p. m.	Time, min. sec.	From in front of pump wheel to river: Speed of engine, r. p. m.	Time, sec.
49.94	29 2.57	49.97	43.50
53.107	26 39.14	53.15	36.76
54.62	25 54.00	54.77	31.27

The average time of travel with different speeds of the engine is shown in Table I., there being 7 tests at each speed for the longer run, and 10 at each speed for the shorter run (through the obstructed outfall portion of the tunnel). The calculated velocity of flow and displacement of the pump wheel are shown in the accompanying diagram. The tests are to be continued in the summer, when the weather conditions are more favorable.

The 24-hour test of the duty of the pumping machinery was started at 11 a. m. on Jan. 22, and the results are shown in Table II.

TABLE II.—DUTY TEST OF PUMPING ENGINE FOR RIVER FLUSHING PLANT; MILWAUKEE.

Total revolutions of engine	76,325
Average revolutions per minute	53
Piston speed, feet per minute	371
Average steam pressure, deducting 5 lbs. for a column of water 11.5 ft. high in the gage pipe	140.206
Average receiver pressure, lbs.	20.036
Average vacuum, ins.	26.410
Total amount of feed-water, lbs.	93,604.54
Average amount of feed-water per hour	3,900.19
Average temperature of feed water	94.5
Temp. of sat. steam at 140.3 lbs. gage pressure (Kent)	360.7
Ave. temperature of superheated steam	497.74
Amount of superheat, degrees F.	137.04
Equivalent for superheat that would have been transmitted to 5,000 lbs. of steam per hour as per guarantee of contractors, degrees F.	106.89
Total coal consumed, lbs.	12,500
Average consumption per hour, lbs.	520.83
Average evaporation per lb. of coal	7.48
Total ashes	1,210
Percentage of ash in coal	9.68
Water displaced per revolution (cu. ft.) as per velocity test	586.77
Water displaced per minute, cu. ft. (a specification requirement)	31,100.84
Total water displacement during test, cu. ft.	44,785,220
Temperature of water pumped, degs. F.	39
Weight of water pumped, lbs. per cu. ft. (Kent)	62.42
Weight of water pumped per revolut'n, lbs.	36,626.1834
Average head of water pumped, feet	3.569
Theoretical displacement of wheel, per revolution without slip, cu. ft.	746.13
Efficiency of wheel, per cent.	78.64
Average Ind. HP. of engine	315.42
HP. of work performed	209.96
Efficiency, pumping machinery, per cent.	66.56
Efficiency of engine, per cent.	87.92
Duty, ft.-lbs., per 1,000 lbs. of steam (a specification requirement)	106,587,951
Duty, ft.-lbs., per 100 lbs. of coal consumed	79,816,928

The tests were made under the direction of Mr. Alex McEachern, erecting engineer for the Allis-Chalmers Co.; Mr. B. W. Perrigo, Assistant City Engineer, and Mr. James T. Hamilton, Assistant Chief Engineer of Pumping Stations.

THE LAW PROHIBITING THE DIVERSION OF THE potable waters of New Jersey to other states has been upheld by the U. S. Supreme Court. The decision was rendered in a suit involving the right of a subsidiary organization of the East Jersey Water Co. to sell water for use in the Borough of Richmond (Staten Island), New York City. It is thought that the decision, combined with that of the state courts, lessens the claims of the large water corporations of New Jersey that they have a monopoly right in the waters of the state.

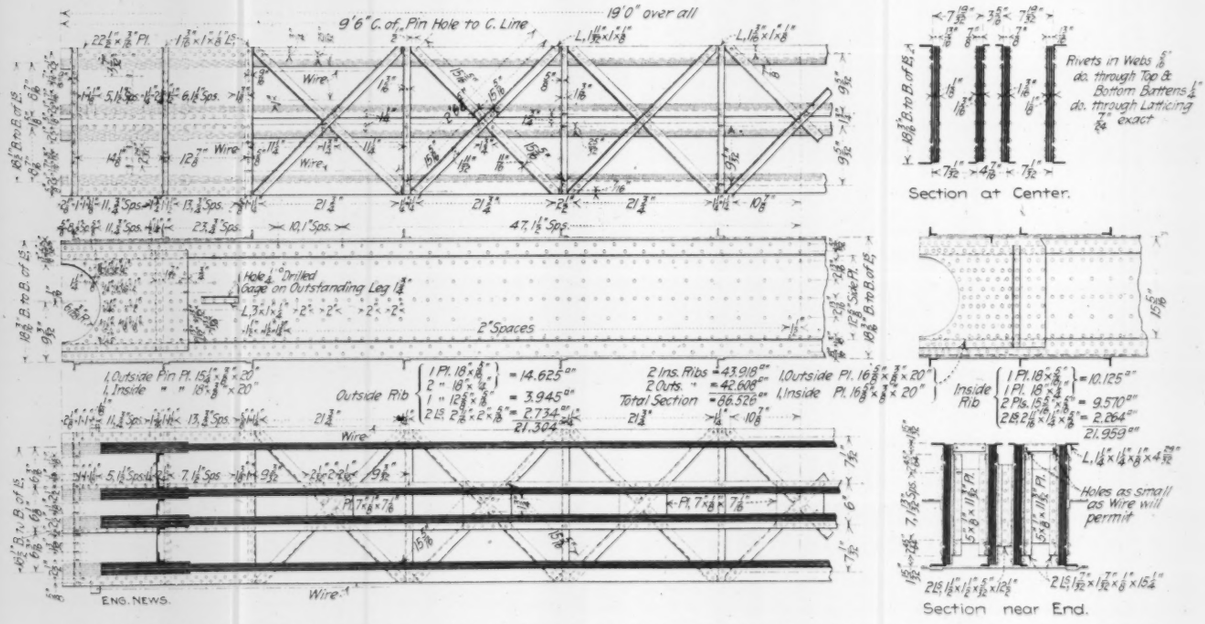


FIG. 1. MODEL OF CHORD A 9 OF THE QUEBEC BRIDGE TO ONE-THIRD LINEAR SCALE, FOR TEST TO DESTRUCTION.

Area of Cross-Section = 86.526 sq. ins. [One-ninth the area of A 9, 780.42 sq. ins. = 86.71 sq. ins.]. Length c. to c. pins, 19 ft.
Length-ratio l/r, for axis parallel to pin, 42; for axis parallel to webs, 35.
Material: Medium steel; Rivets: Soft steel.
Flange angles planed from 3 × 2 × 5/16-in. angle. Lattice angles planed from 1½ × 1½ × ⅛ and 5/32-in. angle. Lattice rivets (7/24-in.) turned from 5/16-in. rivets. All holes drilled.
NOTE: The model differs from Chord A 9 of the Quebec Bridge in lacking the field joint near one end, and in having 12-in. pins instead of 4-in. pins [corresponding to the 12-in. pins of Chord A 9]. Because of the absence of the field-splice, the model has seven bays of lattice both top and bottom, while Chord A 9 had five bays on the upper face and six bays on the lower face.

TESTS OF TWO COMPRESSION CHORD MODELS: THE LARGEST COLUMN TESTS EVER MADE.*

[Appendix 15 of the Quebec Bridge Commission's Report.]

In the course of the Quebec Bridge Inquiry, two tests of compression members were made which are the largest compression tests ever recorded. They have a very intimate relation to the inquiry because the members tested were closely similar to the compression chords of the Quebec Bridge, whose buckling caused the collapse of the bridge. Indeed, one of the two test members was in its main part an exact model of the fatal Chord 9 of the anchor-arm, while the other was similar, differing only in being strengthened at those points where the chief weakness of the Quebec Bridge chords is thought to have been located. It is of special interest to observe that while the first test chord failed at a load whose intensity was not very far above that which wrecked the Quebec Bridge chords, the second test member proved to be some 38% stronger.

These two tests are of remarkable interest and value. We have therefore reproduced the essential parts of the drawings and tables containing their results, in Figs. 1 to 4 herewith. The text-matter of Appendix 15 relating to the tests is reprinted substantially in full, below.

Auxiliary to these compression tests were some tests of rivets in shear and some tension tests of lattice angles like those used in the bottom chords of the Quebec Bridge. The description of these is also given below, but the sketches showing the test-pieces and their results are not reproduced. It appears that the most important showing of the auxiliary tests is that rivet connections have a remarkably uniform strength; that is to say, all the 7/8-in. rivets tested showed single-shear values practically identical, differing by only a very minute percentage, and the double-shear values were sensibly twice as great as the single-shear values.

We have also reproduced from the drawings accompanying the Commission's report the three diagrams Figs. 5, 6 and 7 herewith, which show the distorted shape of three compression chords of the Quebec Bridge as measured on Aug. 27, 1907, two days before the collapse of the bridge. While the scale of transverse deformation in these diagrams is relatively 24 times as large as the longitudinal scale, yet it must be confessed that the conditions revealed by these diagrams are nothing short of startling.

Specially noteworthy is the fact that, while Chord A 9 L was deformed much more seriously than the others, yet two other chords were already buckled in such a way that their failure at a slight increase of load was foreordained. That is, the fall of the bridge is not attributable to a chance, unfortunate deformation of a single chord; at least three chords, at two widely separated points, had reached a condition of immediate danger, and two of them were saved from the stigma of causing the collapse only by the fact that A 9 L had gotten ahead of them in deformation.

The diagrams Figs. 5 to 7, in comparison with

*For other parts of the Quebec Bridge Commission's Report see Eng. News, March 12, 1908, p. 288, Formal Findings: Eng. News, March 19, 1908, p. 307, Abstracts of Report and of Appendices 3 to 11, incl.; Eng. News, April 9, 1908, p. 404, Appendix 13 (Summary of Tests of Large Columns); Eng. News, April 16, p. 421, Appendices 12 and 14 (The Wreck of the Quebec Bridge, and The Stresses in the Bridge).

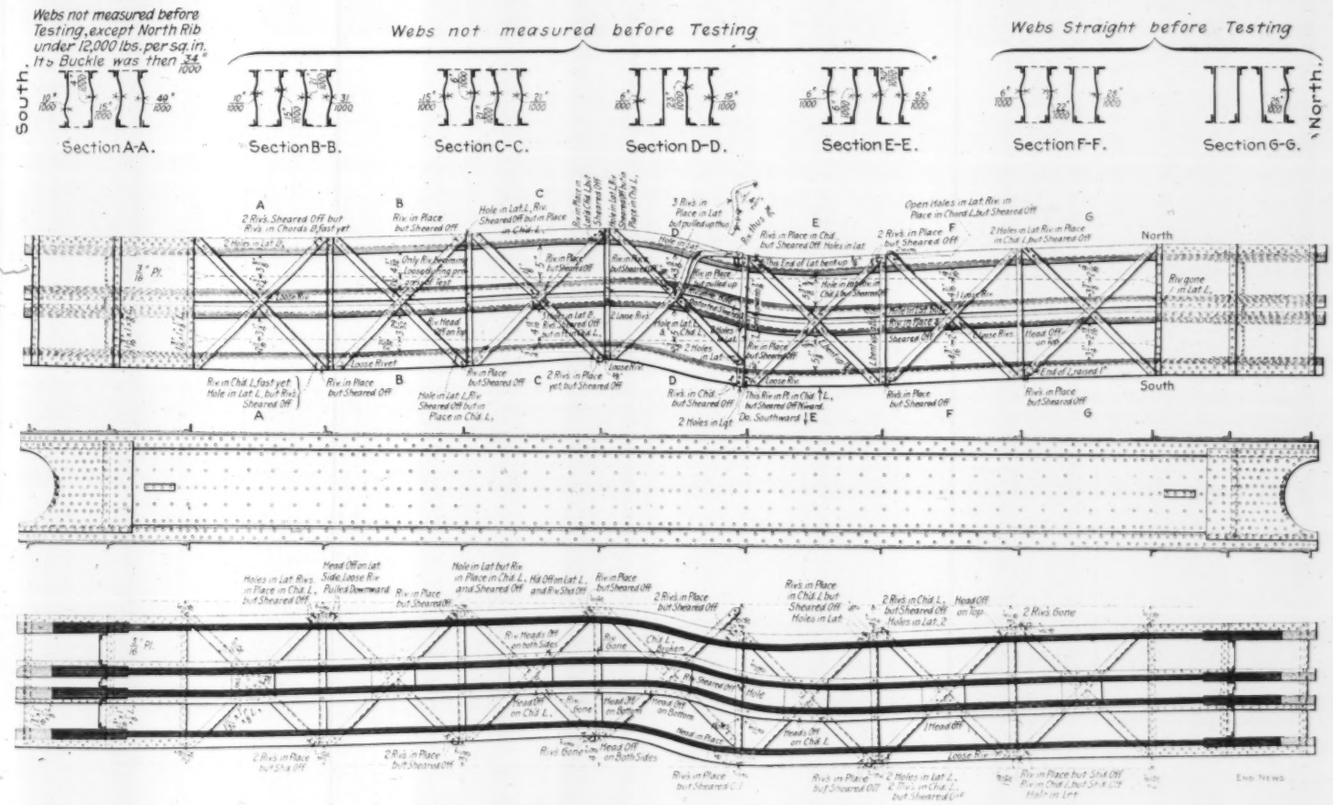


FIG. 2. PLANS, ELEVATIONS AND SECTIONS OF MODEL OF CHORD A 9 AFTER FAILURE.

Indicated Failure Load = 26,850 Lbs. per sq. in. Correction of 17½% Applied by Prof. W. H. Burr on Basis of 1897 Calibration by G. C. Henning Gives Net Failure Load = 22,150 Lbs. per sq. in.

Tested Nov. 21 and 22, 1907, in Phoenix Iron Co.'s hydraulic testing-machine.
Chord Built and Tested for Phoenix Bridge Co.

Chord placed in machine horizontally, pins horizontal. Longitudinal compression was measured on 16-ft. length of outer ribs by means of wooden bar fastened at one end to chord. Lengths of lattice bars tested by trammel-point measurements between center-punch marks on rivet-heads. Deflections, horizontal and vertical, were measured from tight cords stretched close to each upper and lower flange of outer ribs; horizontal deflections as given on drawing; vertical deflections small [not reproduced].

GENERAL RECORD OF TEST.

Indicated load, Lbs. per sq. in.	Duration, min.	Duration, sec.	[All loads were held for 5 mins. before observations were taken.] Remarks.	Compression on 16-ft. length, in ins., N. rib.	Compression on 16-ft. length, in ins., S. rib.
3,000	14	42	No change in lattice-bar lengths. Two N-ribs showed 1/32-in. upward defl. at center.	1/128	1/64
6,000	46	16	No change in lattice-bar lengths. Chords show 1/32-in. upward defl. at center.	+/32	1/32
Load off	40	..	No set observed anywhere.	0	0
9,000	42	..	No change in lattice-bar lengths. Considerable stress in these bars indicated by tapping, and diagonals sound as if one was in tension and the other in compression. Webs are not dished. Three ribs show 1/32-in. upward defl. at center.	3/64	1/16 scant.
12,000	47	..	Slight dishing noticeable, 34/1000-in. at W. end of N. rib. Rivets tight. Lattice-bars as before. Upward defl. of 1/32-in. at center.	1/16 scant.	5/64 scant.
Load off	30	..	No Permanent set. One loose rivet. Ribs remain dished.	0	0
15,000	29	..	Conditions same as under 12,000.	5/64	3/32
16,500	4	..	No observations taken.	..	..
18,000	24	..	Center defl. increased to between 1/32-in. and 1/16-in. Other conditions as before.	3/32	1/8
19,500	3	..	No observations taken.	..	..
Load off	28	..	No permanent set except in length.	1/64	0
21,000	5	..	Length only observed.	1/8	5/32
22,500	7	..	Length only observed.	1/8	5/32
Load off	13	..	No permanent set.	0	0
24,000	14	..	Upward defl. at center for the different ribs = 3/32, 1/32, 3/32, 1/8-in.	9/64	11/64
Load off	28	..	One loose rivet. Upward defl. 0, 1/32, 0, 3/32-in.	0	1/64
25,000	..	..	Test stopped until morning, owing to difficulties with pumps.	..	..
Load off	..	..	One loose rivet. Scale raised in front of both ends of one pin and at one end of the other. No change in length of lattice-bars. Two ribs defl. 1/32-in. upward at center.	1/64	1/64
26,850	..	..	Chord was overloaded by quick action of pump when working for 25,500 lbs. per sq. in. and broke without warning and with a sharp report. Failure due to shearing of lattice rivets as indicated on plan.	abt ½	abt ½

the drawing, Fig. 2, showing the shape of the model test chord after failure, indicate the similarity of the observed distortions to the distortion of the test chord.

The record of the Commission's test is preceded, in Appendix 15, by a reference to the special tests conducted in the course of the construction of the Quebec Bridge:

The Phoenix Iron Co. possesses the most powerful machine for compression tests in existence; unfortunately, some doubt exists as to the accuracy of the records obtained from it. As a result of a series of tests made in 1897, the New York Department of Buildings places its error in compression at 15% in excess; in tension, however, its results seem to be in agreement with those obtained with other machines on similar material. In spite of these doubts, however, this machine has been of invaluable service to the engineering profession.

In evidence, Mr. David Reeves, the president of the Phoenix Bridge Co., states that he had given orders "that all the special tests advised by the Consulting Engineer, Mr. Cooper, or by our own engineers, arising from the unusual size of the bridge be promptly and fully made," so that from the outset the designers of the Quebec Bridge had at their disposal both the equipment for making tests and the authority to use that equipment.

The evidence shows that along certain lines these facilities were by no means neglected, and we are of opinion that had Mr. Cooper and Mr. Szlapka realized how limited is our knowledge of the strength of compression members, they would have made as much use of the testing machine for compression tests as they did in connection with the eyebars.

After mentioning various tests, of pin links, slightly bent eyebars, and erection tackle, the report refers to the large series of tests of eyebars made at Mr. Cooper's direction to determine the deformation of the eye under high loads. This study was summarized in a paper read by Mr. Cooper before the American Society of Civil Engineers (Trans. LVI., 1906, p. 411).

The tests of two model chords and of several rivet connections are then taken up:

Test of a Model of Chord A 9.

After the collapse of the bridge the Phoenix Bridge Co. at its own cost and on its own initiative built and tested the chord shown [by Fig. 1 herewith.—Ed.]. This model chord had, as far as possible, the same relative dimensions as the No. 9 chords of the Quebec Bridge and yet was small enough to be broken in the Phoenix Iron Co.'s testing machine. The test was made on November 21 and 22, 1907, and was under the general direction of Prof. W. H. Burr. By the courtesy of the Phoenix Bridge Co. we are able to give the text of Prof. Burr's report:

Burr's Report on Test of Model Chord No. 1.

* * * The purpose of this test was to secure all possible information regarding the circumstances and method or other features of the failure of that chord which could be disclosed by the test of the model column in question.

* * * All the linear dimensions of the model were exactly one-third of those of the full-size chord section, making the area of cross section (86.526 sq. ins.) one-ninth of that of the full-size member and the volume of metal, with the exception about to be noted, one-twenty-seventh of the original member. This exception arises from the fact that the actual chord member as built, 57 ft. 9/32 in. in length, had a heavy chord joint in it 10 ft. 6 ins., a little more than twice the depth, from one end. Furthermore, the full-size chords were bored for 12-in. pins, and pins of the same diameter were used for the end bearings of the model chord section. It is manifestly impossible to reproduce in a test precisely the conditions existing in the structure at the time of its failure, but it is believed that the end conditions employed in the test and the accurate reproduction by scale of the main dimensions and nearly all the dimensions of the details in the model enable the nearest approach to the actual conditions of the structure to be secured. It is believed that these unavoidable and subordinate departures from the actual conditions of the chord member did not sensibly affect the conditions of failure in the testing machine or the ultimate load carried by the model.

* * * As the plans of both the actual chord [Eng. News, Sept. 19, 1907, p. 320—Ed.] and the model [Fig. 1 herewith—Ed.] show every main and detailed dimension it is not necessary to repeat them here. * * * The linear scale of one-third of the actual dimensions required the rivets to be ¼-in., 5/16-in. and 7/24-in. in diameter, also as shown on the plans, the 7/24-in. rivets being turned down from an original diameter of 5/16-in. Similarly the 2 11/16 × 2-in. and the 2 11/16 × 1¼-in. rib flange angles were planed down from 3 × 2 × 5/16-in. pieces. The lacing angles were also planed from 1½ × 1½ × ⅛-in. and 5/32-in. angle to the dimensions given above. All rivet holes were drilled.

The method of construction of the model was such as to leave it in true and accurate condition. The web-plates were laid off by wood templet, except the pin-plate holes, and drilled, while the pin-plates were drilled from iron templets. The pin-plates were then used as templets for the drilled holes at the ends of the web-plates. One web-plate for each rib thus drilled was used as a drilling templet for the other plates of the same rib, the blank plates being bolted to the drilled plate for this purpose. In the same manner the blank flange angles were bolted to the drilled webs and drilled from the latter as a templet. Rivet holes required for lattice angles were drilled from iron templets but the batten plates first drilled were used for drilling templets after the chord was completely assembled. After the component parts of the ribs were drilled they were taken apart, cleaned, painted and bolted together for riveting. The latter was done both in web and lattice angles with pneumatic hammers. The lattice bars were drilled like the other parts of the model. After the riveting was completed the pin holes were bored and subsequently the ends were faced to proper dimensions in a rotary planer.

All the metal used for the main parts of the model column was medium steel, but soft steel was used for rivets. The steel plates were furnished by the Lukens Iron & Steel Company of Coatesville, Pa., but the angles were supplied and rolled by the Phoenix Iron Co. of Phoenixville, Pa. The rivets were purchased in Philadelphia.

In order that the character of the metal employed might be completely determined, tensile tests were made of both plates and angles, and shearing tests of both the 7/24-in. rivets used in the model and ⅞-in. rivets used in the full-size chord. [The following averages of the results of these tests and of representative specimen tests of the metal used in the chord member 9 as actually built, together with chemical analyses, will represent the tables in the report.—Ed.]

TENSILE TESTS OF PLATES AND ANGLES.

Heat No. or size.	Lbs. per sq. in. Elas. lim.	Lbs. per sq. in. Ult. str.	Per cent. Stretch in 8 ins.	Per cent. Reduction.	Fracture.
1½ × 1½ × ⅛-in. Angles.					
1402	51,025	63,725	23.8	55.1	Silky
3 × 2 × 5/16-in. Angles.					
1402	42,040	62,570	31.6	57.6	Silky
5/16-in. plates; test specimens 1.045-in. wide.					
....	37,810	64,810	29.5	53.1	....
Plates of chord-member 9 as built.					
54½ × ⅞	38,840	60,680	26.5	53.	Silky Cup
54½ × 13/16	41,200	64,890	24.3	50.3	Silky Ang.
Angles of chord member as built.					
8 × 3½ & 8 × 6 × 15/16	38,370	62,350	29.6	49.9	Silky Cup
4 × 3 × ⅜	42,220	66,250	28.2	?	Silky Ang.

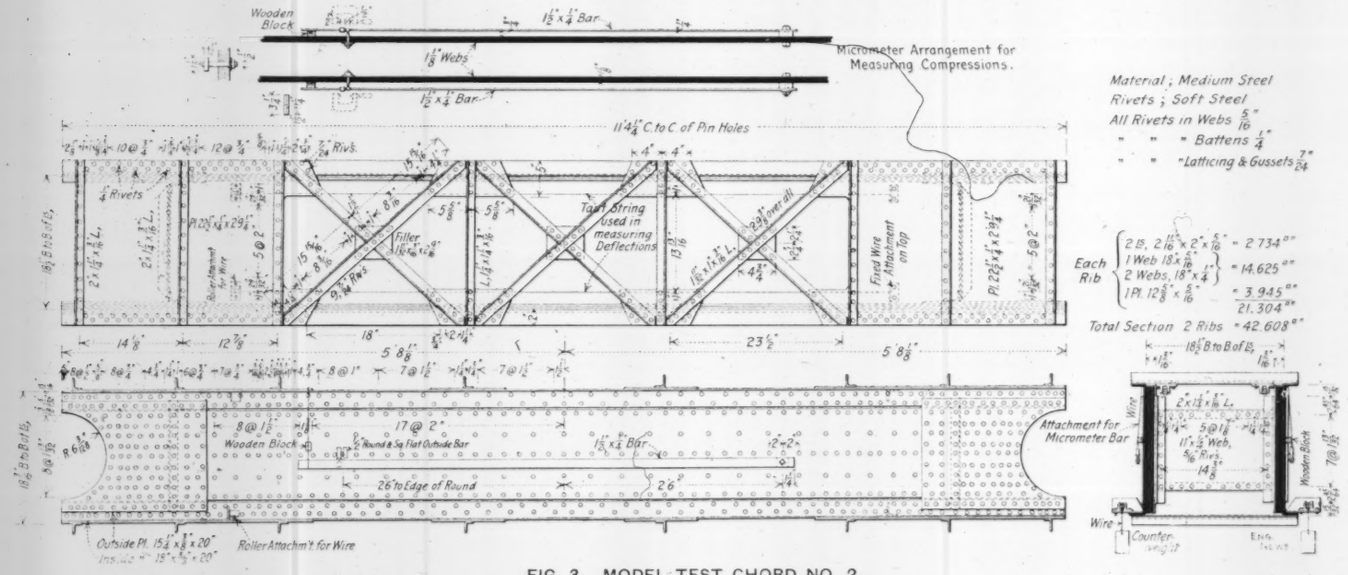

Micrometer Arrangement for Measuring Compressions.
Material; Medium Steel
Rivets; Soft Steel
All Rivets in Webs 5/16"
" " " Battens ¼"
" " " Latticing & Gussets 7/24"
Each Rib
2 L, 2 11/16 × 2" × 5/16" = 2.734 sq in
1 Web 18 × 5/16 = 14.625 sq in
2 Webs, 18" × ¼
1 Pl. 12⅞" × 5/16" = 3.945 sq in
21.304 sq in
Total Section 2 Ribs = 42.608 sq in
11 4¼" C. to C. of Pin Holes
Taut String used in measuring Deflections
Fixed Wire Attachment on top
Attachment for Micrometer Bar
Wooden Block
Wire
Counter weight
Outside Pl. 15¼ × ⅞ × 20' Ins. no. = 15" × ⅝ × 20'
Roller Attachm't for Wire

FIG. 3. MODEL TEST CHORD NO. 2.

Area of Cross-Section Half that of First Model Chord [Fig. 1]; Lattice Angles 50% Larger; Rivets in Lattice Connections Doubled; Intersection of Lattice Diagonals Strengthened by Gusset Plates.

Area of Cross-Section, 42.608 sq. ins. Length c. to c. Pins, 11 ft. 4¼ ins.
Length ratio *l/r*, for axis parallel to pin, 25; for axis parallel to web, 15.
Length-ratio of one rib between lattice connection rivets; axis through its own c. g. parallel to rib, 46.3.
Material: Medium steel, from same heats as material for first model chord. Rivets: Soft steel.
Ribs are identical in make-up with outer ribs of first model chord. The angles and rivets were made from larger sizes in same manner. All holes drilled.

SHEAR TESTS OF RIVETS.

Size of Rivets.	Average of 3 tests lbs. per sq. in.
7/24-in.	59,200
⅞-in.	50,960

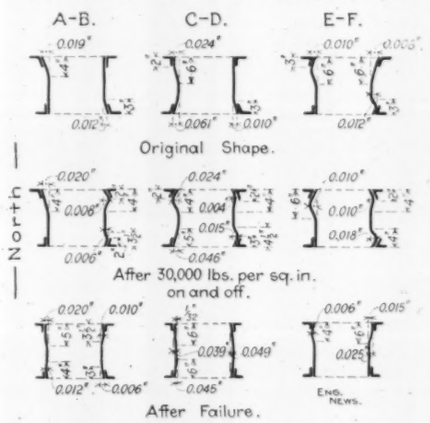
CHEMICAL ANALYSES.

	C.	P.	Mn.	S.
1½ × 1½ × ⅛-in. and 3 × 2 × 5/16-in. angles	.16	.038	.51	.037
5/16 and ¼-in. plates, av.	.22	.020	.41	.023
54½ × ⅞ plates	.17	.01	.46	....
54½ × 13/16 plates	.23	.008	.38	....
8 × 3½ & 8 × 6 × 15/16-in. angles	.165	.046	.38	....
4 × 3 × ⅜ angles	.18	.043	.53	....

The specimen tests of the plates and angles used in the actual chords were selected by me out of a large number so as to give a reasonable and comprehensive view of all and they are fairly representative. It will be observed that the usual effect of rolling thin metal, necessarily finishing at a lower temperature than that in the heavier sections, is apparent in the high elastic limits of 1½ × 1½ × ⅛-in. angles. The same effect, but to a small degree only, is probably discoverable in the 5/16-in. and ⅜-in. angles. This marked effect in the lattice angles of the model column has a distinct bearing upon the final results of the test. A similar general observation, and to a marked degree, applies to the higher unit shearing values of the 7/24-in. rivets as compared with those of the ⅞-in. rivets of the full-size member.

After placing the model column in the machine and under a load producing a stress of 12,000 lbs. per sq. in. it was thought that a buckling or bulging of the web plates was discovered to the extent of .034 in. near the west end of the north rib, but this was found not to increase under further loading. Although measurements of this particular feature were not made before placing the column in the testing machine, continued observations subsequent to the first indicate, I think, conclusively that this particular deformation existed in the column before loading and hence that it had no effect upon the ultimate failure of the column; or in other words, that it was an unavoidable result of the processes employed in the manufacture of the column and was not a true buckling of the plates under loading.

The column was accurately placed in the machine with four fine wires stretched throughout its length in the general plane of the upper flanges and with two similarly placed in relation to the lower flanges. These fine wires, stretched with constant weights, enabled any vertical or horizontal deflections of the tops of the four ribs and the bottoms of the two exterior ribs to be measured by the aid of finely graduated steel scales. Furthermore, longitudinal timber scantlings on the two center lines of the exterior ribs, carrying steel scales at their ends, were used to measure the shortening of the column under loading for 16 ft. of its length to 1/128-in. While these methods of measurements were not so refined as it might be desirable to adopt in an extended series of tests of this nature, they answered well the purposes of this particular investigation, which was not intended so much to determine with refined accuracy all the deformations produced in the test as to discover the main features and methods or other circumstances of failure, so far as possible, which attended the collapse of the full-size chord section.



Prick-punch marks were made in the heads of the rivets of the lattice bars throughout the length of the upper side of the column as it lay in the testing machine, and the distances between these marks were accurately measured at all stages of the test up to failure in order to ascertain the condition of stress in the lattice angles under the progressive loading to which the column was subjected. Furthermore these bars were tapped with a hammer at the same time in order to secure further information as to their condition of stress, as the tone of the resulting sounds might give.

The progressive loading was applied in stages of 3,000 lbs. per sq. in. of cross section of column, beginning with an initial loading of that value. At the end of every other stage of such loading, the column was relieved of stress in order to make observations in that condition. This programme was adhered to up to a stress of 21,000 lbs. per sq. in., when the next increment was made 1,500 lbs. per sq. in., after which the column was freed of load. The remaining programme of loading is shown [below the drawing Fig. 2—Ed.]. After the application of each 3,000 lbs., or finally 1,500 lbs., increment of loading and upon each removal of loading an accurate series of measurements for shortening of the column, for horizontal and vertical deflections at the various panel-points of the latticing, and for the stretching or shortening of the latticing angles was made.

* * * Under a stress of 12,000 lbs. per sq. in. one rivet in a lattice angle at the second center intersection from the west end of the column was found loose, but toward the end of the test it appeared to become less so, the conditions of the center ribs probably becoming such as to give it less opportunity for small motion. Up to the final loading all other rivets appeared to remain in good condition although they were frequently tested with a light hammer.

Actual testing of the column with the application of the first loading began at about 2 p. m., Nov. 21, and it was continued without interruption in the manner set forth in the preceding statements to 11 p. m. of the same day. At that time a load of 25,000 lbs. per sq. in. was reached for a very short time in the endeavor to attain a stress of 25,500 lbs. per sq. in. This endeavor, however, was unsuccessful, in consequence of the leaking of a pump valve (subsequently repaired) to such an extent as to render it impossible to secure the desired pressure in the cylinder of the testing machine.

After having attained the above loading of 25,000 lbs. per sq. in. the progress of the test was interrupted until 10 a. m. of Nov. 22.

At that time instructions were given to load the column to 25,500 lbs. per sq. in., but inadvertence in signaling to attendants at the pump caused the load to reach 26,850 lbs. per sq. in. at which stress the member suddenly failed. This failure was attended by a

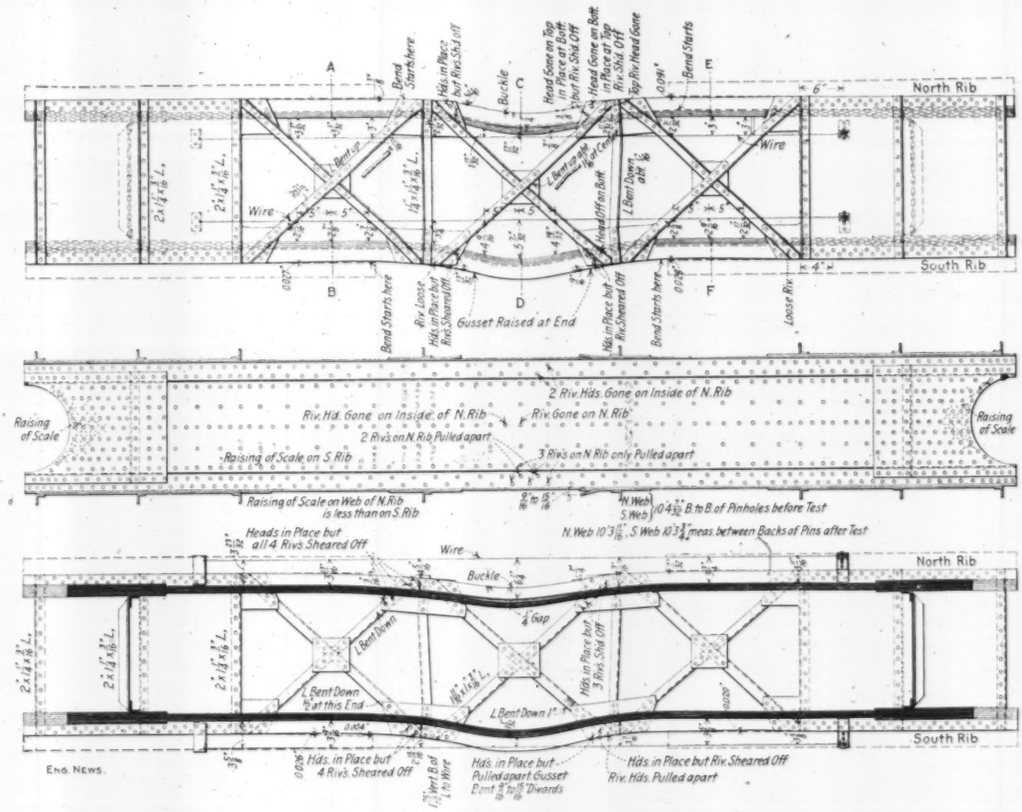

North Rib
South Rib
Wire
Buckle
Gusset Raised at End
2 Riv. Hds. Gone on Inside of N. Rib
Riv. Gone on N. Rib
Raising of Scale
Raising of Scale on S. Rib
3 Rivs. on N. Rib only Pulled apart
Raising of Scale on Web of N. Rib is less than on S. Rib
Heads in Place but all 4 Rivs. Sheared Off
Gap
L Bent Down
L Bent Down at this End
Hds. in Place but 4 Rivs. Sheared Off
Hds. in Place but Pulled apart Gusset
Hds. in Place but Riv. Sheared Off
Riv. Hds. Pulled apart
ENG. NEWS.

FIG. 4. PLANS, ELEVATIONS AND SECTIONS OF MODEL TEST CHORD NO. 2 AFTER FAILURE.

Indicated Failure Load = 37,000 Lbs. per sq. in. Correction of 17½% [as Recommended by Prof. W. H. Burr for First Model Chord] Would Give Net Failure Load = 30,525 Lbs.

Chord Built and Tested for Quebec Bridge Commission.
Tested Jan. 18, 1908, in Phoenix Iron Co.'s hydraulic testing-machine.

Chord placed in machine horizontally, pins horizontal. Longitudinal compression was measured on 5-ft. length [4 ft. $11^{27}/_{32}$ ins. N. rib; 4 ft. $11^{3}/_{4}$ ins. S. rib] by means of 1½ × ¼-in. bar fastened at one end to chord. Lengths of upper lattice-bars were measured before testing, after 15,000 lbs. per sq. in. had been applied and released, and after 20,000 lbs. per sq. in. was applied; no change noticed in either case. Match marks on ends of upper lattice-bars and gusset plates were observed to detect yielding at the various loads; no yielding of any amount was noticed, though slight erratic amounts of tension or compression appeared, shifting location with changing load. Deflections, horizontal and vertical, were measured from wires stretched along upper and lower flange of each rib; vertical deflections negligible [not reproduced]; horizontal deflections as noted on drawing. Test was made for bending of ribs at ends of batten-plates by placing 4-ft. straight-edge along side of rib as shown in plan and measuring distance to face of rib at inner end of straight-edge; the changes under increasing load were small.

GENERAL RECORD OF TEST.

Time.	Load, lbs. per sq. in.	Remarks.	Compressions On 5-ft. length, ins. N. rib.	S. rib.	Betw. inner faces of pins, ins. N. rib.	S. rib.
10 a. m.	15,000	Held for 5 mins.	....	....	....	....
	Load off	After 3 min. took measurements; 3 rivets found loose.	0.	0.	0.013	0.005
11.04	20,000	Tapping of lattice angles gave no result. Of the 3 rivets 2 are now tight.	0.043	0.039	0.114	0.104
11.45	Load off	Pin ends of N. web show raising of scale.	0.002	0.001	0.020	0.010
12.33 p. m.	25,000		0.055	0.052	0.141	0.134
1.00	4,100	Made marks between top lattice-bars and gussets. Testing rivets showed same 3 rivets loose again.				
1.20	28,000	Tapping gave no result.	0.062	0.053	0.162	0.150
1.52	30,000		0.068	0.065	0.180	0.168
2.20	32,000		0.073	0.071	0.199	0.183
2.32	Load off		0.008	0.009	0.055	0.045
3.28	33,000	Loaded to 32,000 and held for a few minutes, then increased to 33,000.	0.077	0.077	0.232	0.221
3.50	34,000	Slight noise noticed.				
3.55	35,000					
4.00	36,000					
4.04	37,000	Distinct noise in center panel. Chord failed under this load at 4.07 p. m.				

quick sharp report and it occurred so suddenly that three observers who were closely watching the column at the time could not discover any sequence in the yielding of the details of the column; the occurrence was so sudden that all failures of details appeared to be absolutely simultaneous.

Aside from the raising of scale on the pin-plates immediately in front of the 12-in. pins, the collapse of the column consisted in the failure by shearing of the majority of the lattice rivets at the central panel of latticing and of a considerable number of other rivets throughout the length of the column in both flanges, leading to the permanent bending to reversed curvature of the four ribs at the same central vicinity accompanied by the violent bending or distortion of the lattice angles and some small dishing of the rib web-plates, all as shown [in Fig. 2]. The ribs were all slightly bent immediately beyond the supporting influence of the battens at each end.

There are certain features of this practically instantaneous failure of the column which are highly significant. As indicated in the preceding statements there were no permanent strains or distortions of any kind discovered or apparently discoverable up to the loading producing failure. This observation is certainly true of every part of the column except the 7/24-in. lattice rivets. If suitable apparatus for refined measurements could have been applied to them some shear distortion might have been observed prior to the final loading. Observations made on the latticing angles showed no permanent stretching or compression of those members prior to failure. The phenomenally high elastic limit of the metal in them shown by the test results in the tabular statement and already remarked upon indicates that they would have exhibited no marked permanent distortion much short of ultimate resistance either as tension or compression members. In point of fact all the circumstances of the test indicated that no main part of the column was stressed up to its elastic limit; in other words, that the entire loading was insufficient to develop more than a part of the elastic resistance of the column as a whole and that if the latticing details had been stronger the column would have carried a greater load before collapsing. The instantaneous failure was clearly due to the fact that the main parts of the column were subject to elastic stress only.

Although it is impossible to correlate accurately the results of this column test with the stress conditions in the actual chord section at the instant of failure, in consequence of the higher elastic qualities of the relatively thin metal in the model column which has already been commented upon, and the presumably greater care which is usually bestowed upon the manufacture of a model member, an approximation of some value may perhaps be made.

The friction of a hydraulic testing machine is known to be considerable, but without recent calibration its amount cannot be confidently stated. The friction of the machine at the Phoenix Bridge Co.'s works was determined some ten or twelve years ago by G. Henning as 17½% of the total load on the piston as indicated by the mercury gage, and this may be accepted provisionally until a further calibration can be made. If this percentage deduction be made from 26,850 lbs. per sq. in., the apparent stress at which the column failed, it will make the compressive stress in the metal 22,110 lbs. per sq. in. The shear tests of the 7/24-in. and ⅞-in. rivets make the average of the latter but 86% of the former. Hence, if the ultimate shearing resistance of the 7/24-in. rivets had been the same as that of the ⅞-in. rivets, the stress on the column producing the failure of the latticing rivets, would have been but 19,014 lbs. per sq. in. of the column. Just what value should be given to the possibly higher excellence of manufacture of the model over that of the full-size column is of course not determinable. It may or may not have sensible value. It is to be noted, however, that, after making such allowance as is practicable for the friction and the increased resistance of the smaller rivets, there is reached an intensity of stress nearly identical with that which existed in the actual chord section at the time of this failure.

It should be carefully observed that the radius of gyration of the normal section of the model column about an axis at right angles to the webs and through their centers, i. e. parallel to the axis of each pin, is 5.43 ins., and 6.52 ins. about a central axis parallel to the webs. Hence, as the column lay in the testing machine, the ratio of its length divided by the horizontal radius of gyration is 35, while the ratio of the same length over the vertical radius of gyration is 42. The column failed therefore in the plane of the greatest radius of gyration. Furthermore, its failure was wholly in a horizontal plane, there being no sensible vertical deflection of the failed column.

The length of the column was such as to place it practically at the limit between short and long columns, as the ordinary column formulas, such as the much used Gordon's and straight-line, are properly applicable when the ratio of length over radius of gyration has values greater than about 40 or possibly a little more. Inasmuch as the ultimate carrying capacity per square inch of section increases as the length of column decreases and as this model column was comparatively short, the latticing required to develop its full load carrying power should be relatively heavy rather than light.

(Signed) Wm. H. Burr.
Consulting Engineer.

Dec. 23, 1907.

The Commissioners were invited to be present and to assist at this test, and the Department of Railways and Canals was represented at it by Mr. C. C. Schneider. The shape of the model chord after the test was finished is shown on drawing [Fig. 2—Ed.], which has been prepared from the blue prints referred to by Prof. Burr.

REMARKS ON TEST OF MODEL CHORD NO. 1

The Commission has the following comments to make concerning this test:

(1) There was little or no indication of failure up

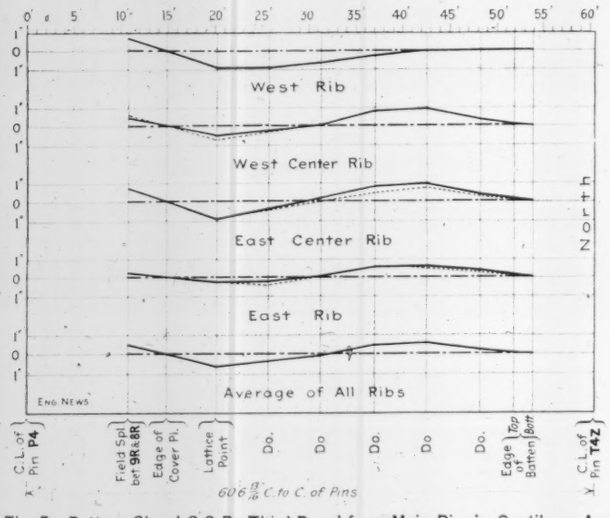


Fig. 5. Bottom Chord C 8 R: Third Panel from Main Pier in Cantilever Arm, East Truss.

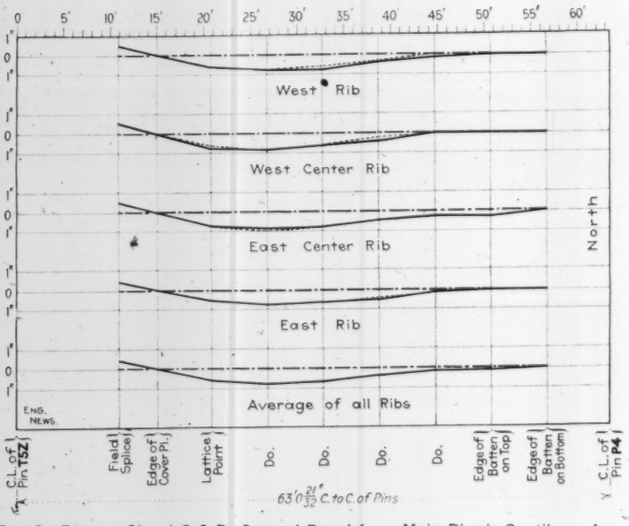


Fig. 6. Bottom Chord C 9 R: Second Panel from Main Pier in Cantilever Arm, East Truss.

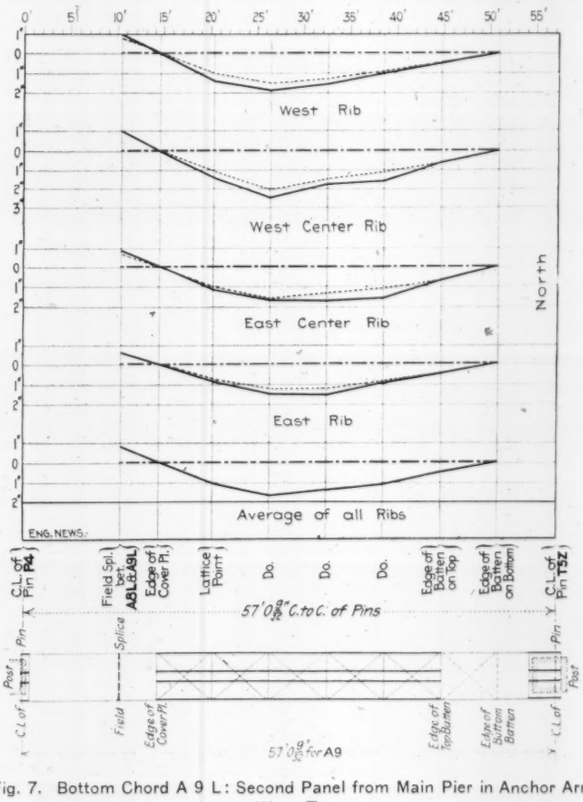


Fig. 7. Bottom Chord A 9 L: Second Panel from Main Pier in Anchor Arm, West Truss.

FIGS. 5, 6 AND 7. DISTORTED SHAPE OF THREE BOTTOM CHORD MEMBERS OF QUEBEC BRIDGE, AS MEASURED TWO DAYS BEFORE THE COLLAPSE.

Measurements taken Aug. 27, 1907.
Transverse scale exaggerated 24 times.
Full line indicates upper edge of rib.
Dotted line indicates lower edge of rib.
Dot-and-dash line indicates reference axis from which deflections were measured.

to the instant at which it occurred. Failure took place with explosive violence, by the shearing of the outer rivets of the latticing.

(2) Messrs. Schneider, Deans and Szlapka were closely watching the chord when the unexpected failure occurred. No one of these engineers could say what connection or detail was the first to give way.

(3) It was noted that the surface scaled at only three outside pin bearings.

(4) The cross lattice bars showed little sign of stress, rivets being sheared only in those bounding the central bay. Theoretically, under a racking stress in the column, these bars would not come into play, the diagonals only being strained, one set in tension and the other set in compression.

(5) The failure of the riveting was systematic. In each lattice bay one diagonal connection failed in tension and the other in compression; and usually both failures occurred at the side of the bay farthest from the nearer end of the column.

It will be noted that both on top and bottom the diagonals in the center bay failed in the opposite manner to that of the corresponding diagonals in all the other bays.

(6) The efficiency of the central connection plate in the bottom latticing is well shown.

(7) Some of the lattice rivets were cut out and found to be partially sheared and there were some slight indications that they had been sheared first on one side and then on the other indicating a reversal of stress in the lattices. That such a reversal would instantly follow the failure of the latticing in the central panel is apparent from the curvature of the chord on one side of the central panel being opposite to that on the other side.

(8) Subsequent investigation has shown that the method adopted for testing the working of the lattice bars was unsatisfactory. It consisted of measurements between center punch marks on the rivet heads and did not include the effect of rivet shear.

In our opinion the load was more evenly and centrally applied in this test than it would be in the case of a chord in ordinary service. In other words under working conditions the failure of the latticing of this model chord would have taken place under a smaller stress.

This is the more probable since, as Prof. Burr points out, the model chord was superior to the bridge chord in both material and workmanship. The difference in material is well shown by the test records included in Prof. Burr's report.

Tests of Rivets.

On Nov. 26 some tests on rivet shear were made by the Phoenix Bridge Co. The results of these tests together with those given by Prof. Burr showed that the rivets used in the bridge would develop an ultimate strength of slightly over 50,000 lbs. per sq. in., and also that the rivets maintained without failure their ultimate strength, even though partially sheared. On January 14 some further tests were made by the Phoenix Bridge Co. which gave similar results. A movement of from 3/16 to ¼ in. apparently took place before actual failure.

This rivet shear offered a reasonable explanation of part of the change in the lengths of the diagonal lattice bars which must have accompanied the distortions of the chords in the bridge which were measured on Aug. 27, 1907 [two days before the collapse. See Figs. 5, 6 and 7—Ed.]. The inspectors had indeed carefully examined the chords and the lattices and reported no evidence of failure, but this rivet shear might easily have escaped their observation; it is noteworthy that no one of the engineers assembled to watch the test of the model chord on Nov. 21 thought of it, and they completely failed to detect such action up to the moment of failure although working under the most favorable conditions. A change in length in addition to the above seemed to be due to the reduced section at the center of one of each pair of diagonal lattice bars.

Test of Model Chord No. 2.

In December the Commission ordered the construction of test chord No. 2 for the purpose of determining the strength of the webs of the design used in the Quebec Bridge. [Fig. 3 shows the construction and dimensions of this chord—Ed.]. It had a section half that of test chord No. 1; the number of rivets in the lattice connections was doubled, the section of the lattice bars was increased 50% and the weak points at their centers were strengthened by the use of connection plates. The webs were of the same section as the outer webs of test chord No. 1. Material from the same heats was used in the manufacture of the two test chords.

This chord was tested at Phoenixville on Jan. 18, 1908. [The shape of the chord after failure is shown by Fig. 4 and the chief observations and measurements are noted below the drawing—Ed.].

It will be noted that this chord failed under a stress of 37,000 lbs. per sq. in., by the buckling of the webs in the center bay, the latticing being sufficiently strong to fully develop the strength of the webs. The nominal strength of the column (the record has to be corrected for an unknown machine error) was slightly less than the elastic limit of the metal in the webs. (See record of specimen tests of the material in Prof. Burr's report).

In this test the column seems to have been loaded evenly and centrally since the latticing was not seriously stressed.

The following notes concerning this test are of interest:

(1) There was some reason to think that chord A 9 L might have been bent sharply at the edge of the cover plate previous to failure. The inclination of its webs to the center line on Aug. 27 was very marked near the 8-9 joint [See Fig. 7—Ed.]. A series of straightedge measurements was made during the test of model chord No. 2 to determine whether any angle developed at the edge of the cover plate as the pressure increased, but no such movement was detected.

(2) During the test the yielding of the lattice rivets was observed by means of match marks upon the lattices and ribs. The results indicate that the pressure was centrally applied and that the lattice-bars were not seriously stressed when these observations were taken. Towards the end of the test the lattice-bars in the end panels were distinctly bowed upwards owing probably to the compression of the webs.

(3) Scaling at the pin bearings was observed, but this was about the same at all four surfaces at the bearings and did not indicate that there was any racking strain on the chord.

(4) The dishing of the webs during the test is shown on the drawing [not all sections and loads are reproduced—Ed.].

(5) * * * There were practically no horizontal or vertical movements of the chord webs with reference to the chord ends to which the reference wires were attached.

Tests of Full-Size Lattice Angles and Rivets.

On Jan. 20, 1908, the Commission made three tests on full-size lattice bars of the chord No. 9 design. The tests were made in the laboratory of Messrs. Wm. Sellers & Co., Philadelphia, and the results obtained under the skillful handling of Mr. Backstrom may be accepted without question. The purpose of these tests was to determine the strength of the lattice-bars and the amount of yielding of the various parts of the length as the tension increased. * * * In each case failure took place at the center of the lattice-bar, and the accompanying table is of interest.

It will be noted from the above that the riveting was sufficiently strong to develop the full strength of the cut lattice-bar. Yielding took place simultaneously in the rivet connections and in the reduced section at the center of the bar. As the first test chord failed by shearing of the rivet connections and as no indications of failure at the center of the lattice-bars were noticed, the result of the tests on the full-size lattice-bars was unexpected.

A series of tests on the lattice-bars of test chord No. 1 was made by direction of the Commission on Jan. 23, 1908. The small testing-machine belonging to the Phoenix Iron Co. was used, but as this was not well equipped for the work, the results are not wholly satisfactory. * * * Up to the moment of failure there was little yielding either of the rivets or of the reduced central portion, and the failure took place in each case by rivet shear. The results of the specimen tests on the material for the angles are given in Prof. Burr's report.

The following table gives the results of the individual tests upon the 7/24-in. rivets.

RESULTS OF LATTICE AND RIVET TESTS.

3 full-size bars, Wm. Sellers & Co., Jan. 20, 1908.	3 short sections of full-size bars Phoenix Bridge Co., Jan. 21, 1908.	4 tests on two ⅞-in. rivets in single shear, Phoenix Bridge Co., Nov., 26, 1907.	2 tests on two ⅞-in. rivets in single shear, Phoenix Bridge Co., Jan. 14, 1908.	3 tests upon two ⅞-in. rivets in double shear, Phoenix Bridge Co., November, 1907.
Ultimate load in lbs.	Ultimate load in lbs.	Ultimate load in lbs.	Ultimate load in lbs.	Ultimate load in lbs.
60,100	61,100	63,000	62,500	121,000
59,800	62,000	63,000	63,100	122,000
59,500	60,700	63,800	.....	123,800
.....	.....	64,700	.....	.....

TESTS ON 7/24-IN. RIVETS.

At Phoenixville, Nov., '07. Two Rivets in Double Shear. Ultimate load in lbs.	At Phoenixville, Jan. 21, '08. Two Rivets in Single Shear. Ultimate load in lbs.
16,000	7,500
15,600	8,700
16,000	9,000
Av. 15,900	8,400

On Jan. 31, the Commission made some tests at the laboratory of Messrs. Wm. Sellers & Co. to determine the slip of rivets connecting parts under compression. [Two 3 x 2-in. flat bars were riveted along the middle of one side of the web of a 6-in. I-beam 13½ ins. long, by two ⅞-in. rivets each, leaving a space of about 2 ins. between their adjoining ends. The I-beam bent under the eccentric loading before the riveting gave way. The tables of results are not reproduced.—Ed.].

Under light loads the slip was much the same as in tension tests and as the loading increased the web of the I-beam and not the riveting gave way.

LENGTH OF RECORDS NECESSARY FOR DETERMINING STREAM FLOW.

By JOHN C. HOYT,* Assoc. M. Am. Soc. C. E.

In planning for enterprises involving the use of water in streams, the engineer has to base his estimates of the quantity and distribution of the flow that may be expected in the future on records for past years. The flow of a stream is a constantly changing quantity and neither the total quantity or its distribution for any year will probably ever be the same in any other year. Therefore, the question arises as to how long a series of observations is necessary in order to determine what conditions may be expected in the future.

By the study of long series records of flow which are now available for various streams in the United States an idea of the necessary period over which observations should be carried can be obtained. Such a study based on the records (Tables I. to III.) of the Ohio River at Wheeling, W. Va. (1884 to 1905) drainage area 23,800 sq. mi., the Tennessee River near Chattanooga, Tenn. (1874 to 1906) drainage area 21,400 sq. mi., and the Sudbury River at Framingham, Mass. (1875 to 1906), drainage area 75.2 sq. mi., will be of interest to the users of stream flow data.

For these stations the mean annual flow has been tabulated for 1, 5, and 10-year periods, and

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TABLE I.—RUN-OFF OF THE OHIO RIVER AT WHEELING, W. VA.

One-year period.				Five-year period.				Ten-year period.			
		Difference from 21-yr. mean.				Difference from 21-yr. mean.				Difference from 21-yr. mean.	
Year.	Mean run-off, ins.	Ins.	Per cent. of mean.	Years.	Mean run-off, ins.	Ins.	Per cent. of mean.	Years.	Mean run-off ins.	Ins.	Per cent. of mean.
1884-85	17.82	—4.86	—21								
1885-86	21.18	—1.50	— 6.6								
1886-87	22.39	—0.29	— 1.3								
1887-88	17.46	—5.22	—23								
1888-89	24.34	+1.66	+ 7.3	1884-89	20.64	—2.04	— 9.0				
1889-90	34.20	+11.52	+51	1885-90	23.91	+1.23	+ 5.4				
1890-91	30.64	+7.96	+35	1886-91	25.81	+3.13	+14				
1891-92	22.09	—0.59	— 2.6	1887-92	25.75	+3.07	+13.5				
1892-93	21.52	—1.16	— 5.1	1888-93	26.56	+3.88	+17				
1893-94	18.03	—4.65	—21	1889-94	25.30	+2.62	+12	1884-94	22.97	+0.29	+1.3
1894-95	16.29	—6.39	—28	1890-95	21.71	—0.97	— 4.3	1885-95	22.81	+0.13	+0.6
1895-96	19.71	—2.97	—13	1891-96	19.53	—3.15	—14	1886-96	22.67	—0.01	—0.04
1896-97	22.59	—0.09	— 0.4	1892-97	19.63	—3.05	—13	1887-97	22.69	+0.01	+0.04
1897-98	24.61	+1.93	+ 8.5	1893-98	20.25	—2.43	—11	1888-98	23.40	+0.72	+3.2
1898-99	22.88	+0.20	+ .9	1894-99	21.22	—1.46	— 6.4	1889-99	23.25	+0.57	+2.5
1899-00	17.64	—5.04	—22	1895-00	21.49	—1.19	— 5.2	1890-00	21.60	—1.08	—4.8
1900-01	24.55	+1.87	+ 8.2	1896-01	22.45	—0.23	— 1.0	1891-01	20.99	—1.69	—7.5
1901-02	24.66	+1.98	+ 8.7	1897-02	22.87	+0.19	+ 0.8	1892-02	21.25	—1.43	—6.3
1902-03	27.09	+4.41	+19	1898-03	23.36	+0.58	+ 2.6	1893-03	21.80	— .88	—3.9
1903-04	25.35	+2.67	+12	1899-04	23.86	+1.18	+ 5.2	1894-04	22.55	— .18	—0.6
1904-05	21.19	—1.49	— 6.6	1900-05	24.57	+1.89	+ 8.3	1895-05	23.03	+ .35	+1.5

Run-off for 21 years: Mean, 22.68; Max., 34.20; Min., 16.29.

Note: The mean run-off is in depth in inches over the drainage area. The year used is from Oct. 1 to Sept. 30.

TABLE II.—RUN-OFF OF THE TENNESSEE RIVER AT CHATTANOOGA, TENN.

One-year period.				Five-year period.				Ten-year period.			
		Difference from 32-yr. mean.				Difference from 32-yr. mean.				Difference from 32-yr. mean.	
Year.	Mean run-off, ins.	Ins.	Per cent. of mean.	Years.	Mean run-off, ins.	Ins.	Per cent. of mean.	Years.	Mean run-off ins.	Ins.	Per cent. of mean.
1874-75	26.00	+0.77	+ 3.1								
1875-76	30.01	4.78	+19								
1876-77	20.15	—5.08	—20								
1877-78	19.06	—6.17	—24								
1878-79	**21.21**	—4.02	—16	1874-79	**23.29**	—1.94	— 7.7				
1879-80	27.08	+1.85	+ 7.3	1875-80	23.50	—1.73	— 6.9				
1880-81	22.65	—2.58	—10	1876-81	22.03	—3.20	—13				
1881-82	37.10	+11.87	+47	1877-82	25.42	+0.19	+ 0.8				
1882-83	24.63	—0.60	— 2.4	1878-83	26.53	+1.30	+ 5.2				
1883-84	32.74	+7.51	+30	1879-84	28.84	+3.61	+14	1874-84	26.06	+0.83	+3.3
1884-85	17.96	—7.27	—29	1880-85	27.02	+1.79	+ 7.1	1875-85	25.26	+0.03	+0.1
1885-86	35.79	+10.56	+42	1881-86	29.64	+4.41	+17	1876-86	25.84	+0.61	+2.4
1886-87	25.50	+0.27	+ 1.1	1882-87	27.32	+2.09	+ 8.3	1877-87	26.37	+1.14	+4.5
1887-88	22.40	—2.83	—11	1883-88	26.88	+1.65	+ 6.5	1878-88	26.71	+1.48	+5.9
1888-89	26.74	+1.51	+ 6.0	1884-89	25.68	+0.45	+ 1.8	1879-89	27.26	+2.03	+8.0
1889-90	28.18	+2.95	+12	1885-90	27.72	+2.49	+ 9.9	1880-90	27.37	+2.14	+8.5
1890-91	32.45	+7.22	+29	1886-91	27.05	+1.82	+ 7.2	1881-91	28.35	+3.12	+12
1891-92	26.45	+1.22	+ 4.8	1887-92	27.24	+2.01	+ 8.0	1882-92	27.28	+2.05	+8.0
1892-93	23.98	—1.25	— 5	1888-93	27.56	+2.33	+ 9.2	1883-93	27.22	+1.99	+7.9
1893-94	17.27	—7.96	—32	1889-94	25.67	+0.44	+ 1.7	1884-94	25.67	+0.44	+1.7
1894-95	20.28	—4.95	—20	1890-95	24.09	—1.14	— 4.5	1885-95	25.90	+0.67	+2.7
1895-96	18.24	—6.99	—28	1891-96	21.24	—3.99	—16	1886-96	24.15	—1.08	—4.3
1896-97	27.96	+2.73	+11	1892-97	21.55	—3.68	—15	1887-97	24.40	—0.83	—3.3
1897-98	18.93	—6.30	—25	1893-98	20.54	—4.69	—19	1888-98	24.05	—1.18	—4.7
1898-99	31.00	+5.77	+23	1894-99	23.28	—1.95	— 7.7	1889-99	24.47	—0.76	—3.0
1899-00	18.97	—6.26	—25	1895-00	23.02	—2.21	— 8.8	1890-00	23.55	—1.68	—6.7
1900-01	30.59	+5.36	+21	1896-01	25.49	+0.26	+ 1.0	1891-01	23.37	—1.86	—7.4
1901-02	26.86	+1.63	+ 6.5	1897-02	25.27	+0.04	+ 0.2	1892-02	23.41	—1.82	—7.2
1902-03	27.66	+2.43	+ 9.6	1898-03	27.02	+1.79	+ 7.1	1893-03	23.78	—1.45	—5.8
1903-04	25.88	+0.65	+ 2.6	1899-04	25.99	+0.76	+ 3.0	1894-04	24.64	—0.59	—2.3
1904-05	19.57	—5.66	—22	1900-05	26.11	+0.88	+ 3.5	1895-05	24.57	—0.66	—2.6
1905-06	23.94	—1.29	— 5	1901-06	24.78	—0.45	— 1.8	1896-06	25.14	—0.09	—0.4

Run-off for the 32 years; Mean, 25.23; Max., 37.10; Min., 17.27.

Note: The mean run-off is in depth in inches over the drainage area. The year used is from Oct. 1 to Sept. 30.

TABLE III.—RUN-OFF OF THE SUDBURY RIVER AT FRAMINGHAM, MASS.

One-year period.				Five-year period.				Ten-year period.			
		Difference from 32-yr. mean.				Difference from 32-yr. mean.				Difference from 32-yr. mean.	
Year.	Mean run-off, sec. ft. per sq. mi.	Sec. ft. per sq. mi.	Per cent. of mean.	Years.	Mean run-off, sec. ft. per sq. mi.	In sec. ft. per sq. mi.	Per ct. of mean.	Years.	Mean sec. ft. per sq. mi.	In sec. ft. per sq. mi.	Per ct. of mean.
1875	1.504	—.144	8.7								
1876	1.756	+.108	6.6								
1877	1.878	+.230	14								
1878	2.246	+.598	36								
1879	1.383	—.265	16	1875-79	1.753	+.105	6.4				
1880	0.895	—.753	46	1876-80	1.632	—.016	1.0				
1881	1.515	—.133	8.1	1877-81	1.583	—.065	3.9				
1882	1.334	—.314	19	1878-82	1.475	—.173	10				
1883	0.824	—.824	50	1879-83	1.190	—.458	28				
1884	1.747	+.099	6.0	1880-84	1.263	—.385	23	1875-84	1.508	—.140	8.5
1885	1.393	—.255	15	1881-85	1.393	—.285	17	1876-85	1.497	—.151	9.2
1886	1.682	+.034	2.1	1882-86	1.396	—.252	15	1877-86	1.490	—.158	9.6
1887	1.785	+.137	8.3	1883-87	1.488	—.162	9.8	1878-87	1.480	—.168	10
1888	2.626	+.978	59	1884-88	1.847	+.199	12	1879-88	1.518	—.130	7.9
1889	2.140	+.492	30	1885-89	1.925	+.277	17	1880-89	1.594	—.054	3.3
1890	1.989	+.341	21	1886-90	2.044	+.396	24	1881-90	1.704	+.056	3.4
1891	2.034	+.386	23	1887-91	2.115	+.467	28	1882-91	1.755	+.107	6.5
1892	1.200	—.439	27	1888-92	2.000	+.352	21	1883-92	1.743	+.095	5.8
1893	1.604	—.044	2.7	1889-93	1.795	+.147	8.9	1884-93	1.821	+.173	10
1894	1.192	—.456	28	1890-94	1.606	—.042	2.5	1885-94	1.765	+.117	7.1
1895	1.782	+.134	8.1	1891-95	1.564	—.084	5.1	1886-95	1.804	+.156	9.5
1896	1.576	—.072	4.4	1892-96	1.473	—.175	11	1887-96	1.794	+.146	8.9
1897	1.533	—.115	7.0	1893-97	1.537	—.111	6.7	1888-97	1.768	+.120	7.3
1898	2.248	+.600	36	1894-98	1.666	+.018	1.1	1889-98	1.731	+.083	5.0
1899	1.51	—.138	8.4	1895-99	1.730	+.082	5.0	1890-99	1.668	+.020	1.2
1900	1.674	+.026	1.6	1896-00	1.708	+.060	3.6	1891-00	1.636	—.012	.7
1901	2.076	+.428	26	1897-01	1.808	+.160	9.8	1892-01	1.640	—.008	.5
1902	1.764	+.116	7.0	1898-02	1.854	+.206	12	1893-02	1.696	+.048	2.9
1903	1.841	+.193	12	1899-03	1.773	+.125	7.6	1894-03	1.720	+.072	4.4
1904	1.441	—.207	13	1900-04	1.759	+.111	6.7	1895-04	1.744	+.096	5.8
1905	1.230	—.418	25	1901-05	1.670	+.022	1.3	1896-05	1.689	+.041	2.5
1906	1.331	—.317	19	1902-06	1.521	—.127	7.7	1897-06	1.665	+.017	1.0

Run-off for 32 years: Mean, 1.648; Max., 2.626; Min., 0.824.

TABLE IV.—RUN-OFF OF THE COLORADO RIVER AT YUMA, ARIZ.

One-year period.				Five-year period.				Ten-year period.			
		Difference from 28-yr. mean.				Difference from 28-yr. mean.				Difference from 28-yr. mean.	
Year.	Mean discharge in. sec. ft.	In. sec. ft.	Per ct. of mean.	Years.	Mean discharge, in sec. ft.	In sec. ft.	Per ct. of mean.	Years.	Mean discharge in sec. ft.	In sec. ft.	Per ct. of mean.
1879	2,730	—7,370	—73								
1880	5,610	—4,490	—44								
1881	5,970	—4,130	—41								
1882	4,720	—5,380	—53								
1883	6,850	—3,250	—32	1879-83	5,180	—4,900	—48				
1884	14,500	+4,400	+44	1880-84	7,530	—2,600	—26				
1885	6,350	—3,750	—37	1881-85	11,700	+1,600	+16				
1886	6,870	—3,230	—32	1882-86	7,850	—2,250	—22				
1887	5,140	—4,960	—49	1883-87	7,940	—2,160	—21				
1888	4,640	—5,460	—54	1884-88	7,500	—2,600	—26	1879-88	6,300	—3,800	—38
1889	4,680	—5,420	—54	1885-89	5,500	—4,600	—46	1880-89	6,500	—3,600	—36
1890	10,400	+ 300	+ 3	1886-90	6,300	—3,800	—38	1881-90	7,000	—3,100	—31
1891	10,000	— 100	— 1	1887-91	7,000	—3,100	—31	1882-91	7,400	—2,700	—27
1892	10,200	+ 100	+ 1	1888-92	8,000	—2,100	—21	1883-92	8,000	—2,100	—21
1893	7,860	—2,240	—22	1889-93	8,600	—1,500	—15	1884-93	8,060	—2,040	—20
1894	7,430	—2,670	—26	1890-94	9,200	— 900	— 9	1885-94	7,400	—2,700	—27
1895	9,710	— 390	— 4	1891-95	9,040	—1,060	—10	1886-95	7,700	—2,400	—24
1896	8,980	—1,120	—11	1892-96	8,800	—1,300	—13	1887-96	7,900	—2,200	—22
1897	12,400	+2,300	+23	1893-97	9,300	— 800	— 8	1888-97	8,600	—1,500	—15
1898	9,080	—1,020	—10	1894-98	9,500	— 600	— 6	1889-98	9,070	—1,030	—10
1899	12,200	+2,100	+21	1895-99	10,500	+ 400	+ 4	1890-99	9,800	— 300	— 3
1900	9,400	— 700	— 7	1896-00	10,400	+ 300	+ 3	1891-00	9,700	— 400	— 4
1901	11,700	+1,600	+16	1897-01	11,000	+ 900	+ 9	1892-01	9,900	— 200	— 2
1902	11,000	+ 900	+ 9	1898-02	10,700	+ 600	+ 6	1893-02	10,000	— 100	— 1
1903	15,600	+5,500	+55	1899-03	12,000	+1,900	+19	1894-03	10,800	+ 700	+ 7
1904	13,900	+3,800	+38	1900-04	12,300	+2,200	+22	1895-04	11,400	+1,300	+13
1905	27,300	+17,200	+171	1901-05	15,900	+5,800	+58	1896-05	13,200	+3,100	+31
1906	26,900	+16,800	+166	1902-06	18,900	+8,800	+88	1897-06	14,900	+4,800	+48

Run-off for 28 years: Mean, 10,100; Max., 27,300; Min., 2,730.

the difference of each of these values from the grand mean for the whole period has been determined and expressed both in the unit of measurement and in percentage of variation from the mean. The 5 and 10-year periods have been taken as follows: First to fifth year, inclusive, second to sixth year, inclusive, etc.; first to tenth year, inclusive, etc., etc. An examination of these tables shows: (1) the values represented by the one-year period vary so much from the grand mean that estimates based upon one year alone are liable to be in great error; (2) while the 5-year period gives in the majority of cases values within 10% of the grand mean, there are years when the difference may be even 20 or 30%. The probability of the 5-year period containing both a year of average low water and average high water, is small; (3) the 10-year period with few exceptions gives values within less than 10% of the grand mean. Furthermore, there occurs in practically each of the 10-year groups a year of average low water and also of average high water. While this low and high may not be the extreme, it gives nevertheless the mean condition which may be expected with the exception of the abnormal year, which as a rule only occurs once in many years. It is believed, therefore, that a 10-year period will give a fair idea of the flow that may be expected on any eastern stream.

These deductions are drawn from records on streams in the humid region and do not hold for streams in the arid and semi-arid regions, where the flow is usually very erratic and the range is very great from year to year, as shown by Table IV., for the Colorado River at Yuma, Ariz., with a drainage area 225,000 sq. mi.

That caution should be exercised in the design of works involving the use of water is shown very forcibly in the great yearly variation in the flow at these stations. Records extending over a period of years are the only safe guide in making such estimates, and until such estimates are available for the country at large many failures in connection with hydraulic development are inevitable. Illustrations of these failures may be seen in all parts of the country and should be object lessons to both the investor and the engineer in plans for future work.

THE ECONOMY OF ONE POWERFUL LOCOMOTIVE in place of two lighter engines to haul the same weight of train has been tested on the Northern Ry. of France. According to a paragraph in the engineering supplement of the London "Times" the trains were of the standard weight of 950 tons, and the tests covered runs of 12,400 miles. The powerful engine was of the Mallet duplex type, with high-pressure cylinders driving one group of axles and low-pressure cylinders driving a steam truck. The other engines were of the 0-6-0 and 0-8-0 types, coupled together. The cost for fuel, oil, maintenance and wages for six months was $761 for the one and $1,041 for the two engines, or a saving of $280. The first cost was $23,280 for the Mallet engine, and $17,900 and $12,220 for the two engines.

AN INSPECTION OF SCHOOL BUILDINGS in New York City by Mr. C. B. J. Snyder, Superintendent of School Buildings, shows the existence of 429 such buildings without proper safeguards against fire and panic. These buildings are distributed as follows: Manhattan, 112; The Bronx, 23; Brooklyn, 164; Queens, 90; Richmond, 40. Apparently, in classifying buildings as unsafe, a very high standard was set, inasmuch as Mr. Snyder is reported as having stated that to replace all the buildings with first-class fireproof structures would cost upwards of $60,000,000, but that such an expenditure at once is unnecessary since it is not absolutely necessary to abandon buildings of brick with wood floor beams. On the subject of doors swinging inward and outward, Mr. Snyder says:

The classroom doors of the schools of the old city of New York formerly opened out into the corridor, but it was found that they were thus the greatest contributing cause in the lack of control of the pupils by the teachers when an alarm was given, as they had difficulty in retaining the children in the rooms.

As a result all classroom doors of the school buildings were then changed to swing into the classrooms and the placing of two doors to a room was avoided wherever possible.

There is much evidence to show the wisdom of this plan, one being the statement made to me by one of the teachers at the ill-fated Collinwood school, where the doors opened out into the hall, that she could have saved all her children but that all ran by her out of the door into the corridor; she saved only those she could hold in her arms.

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President Roosevelt's veto last week of a bill to extend the time for the construction of a dam across Rainy River and his demand that every such grant of a water-power franchise shall carry with it recognition of the rights of the public has met with hearty and well-nigh universal approbation in the public press. In this particular case the promoters got their original charter ten years ago, which required them to begin work in one year and complete it in three years. They have come back to Congress again and again to have the time extended; and Congress, which is so overwhelmed with work that it cannot take action on important public measures, has always found time to grant their request.

It is certainly the height of absurdity to give away franchise rights for the development of natural resources, and permit the holder of the franchise to do nothing year after year. Actually such franchises are often obtained by promoters and speculators, who have neither the intention or ability to utilize the franchise. They simply play dog-in-the-manger and prevent the actual development until some one comes along willing to buy up their franchise. Part of the President's veto message is so important that it deserves quotation in full:

> The public must retain the control of the great waterways. It is essential that any permit to obstruct them, for reasons and on conditions that seem good at the moment, should be subject to revision when changed conditions demand. The right reserved by Congress to alter, amend, or repeal is based on this principle; but actual experience of what happens with indeterminate public-utility franchises proves that they are in the vast majority of cases practically perpetual. Each right should be issued to expire on a specified day without further legislative, administrative, or judicial action.
>
> Every permit to construct a dam on a navigable stream should specially recognize the right of the Government to fix a term for its duration and to impose such charge or charges as may be deemed necessary to protect the present and future interests of the United States in accordance with the act of June 21, 1906.
>
> The provision for a charge is of vital importance. The navigability of every inland waterway, and of all connected or connectible inland waterways as a whole, should be improved for the purposes of interstate and foreign commerce upon a consistent unified plan by which each part should be made to help every other part. One means available for the improvement of navigation at a particular point on any river may be a dam creating a slack-water pool of sufficient depth. Such a dam may, in many cases, develop power of sufficient value to pay in whole or in part for the improvement of navigation at that point, and if there is any surplus it can be spent upon improvements at other points in acordance with the general plan.
>
> The income derivable from this source would materially aid in the complete improvement of our navigable waters, for which there is now such crying need. The Chief of Engineers of the Army reports that the bills pending at this session of Congress permit the construction of dams in navigable streams capable of developing over 1,300,000 HP. These rivers run every hour in the day and every day in the year. To develop this amount of power would, under average conditions, require about 25,000,000 tons of medium-quality coal every year. This natural wealth is the heritage of the people. I see no reason for giving it away, though there is every reason for not imposing conditions so burdensome as to prevent the utilization of the power.
>
> We are now at the beginning of great development in water power. Its use through electrical transmission is entering more and more largely into every element of the daily life of the people. Already the evils of monopoly are becoming manifest; already the experience of the past shows the necessity of caution in making unrestricted grants of this great power.
>
> The present policy pursued in making these grants is unwise in giving away the property of the people in the flowing waters to individuals or organizations practically unknown, and granting in perpetuity these valuable privileges in advance of the formulation of definite plans as to their use. In some cases the grantees apparently have little or no financial or other ability to utilize the gift, and have sought it merely because it could be had for the asking.
>
> In place of the present haphazard policy of permanently alienating valuable public property we should substitute a definite policy along the following lines:
>
> First. There should be a limited or carefully guarded grant in the nature of an option or opportunity afforded within reasonable time for development of plans and for execution of the project.
>
> Second. Such a grant or concession should be accompanied in the act making the grant by a provision expressly making it the duty of the designated official to annul the grant if the work is not begun or plans are not carried out in accordance with the authority granted.
>
> Third. It should also be the duty of some designated official to see to it that in approving the plans the maximum development of the navigation and power is assured, or at least that in making the plans these may not be so developed as ultimately to interfere with the better utilization of the water or complete the development of the power.
>
> Fourth. There should be a license fee or charge which, though small or nominal at the outset, can in the future be adjusted so as to secure a control in the interest of the public.
>
> Fifth. Provision should be made for the termination of the grant or privilege at a definite time, leaving to future generations the power or authority to renew or extend the concession in accordance with the conditions which may prevail at that time.

The bill to create Appalachian forest reserves is held up while the lawyers in Congress discuss the question whether the Federal Government has the authority under the constitution to condemn lands for purposes of forest preservation. It is contended that the relation of forest preservation to the flow of water in navigable rivers is too remote and uncertain a matter on which to base legal authorization for the condemnation of lands by the Federal Government.

It is well that this constitutional question should be settled—and settled right—for it is to come forward with increasing frequency in a great variety of cases. Is the constitution—the charter on which our whole governmental organization rests—broad enough to permit the Federal Government to act for the public welfare where action is urgently needed and where the states themselves can do the work only with difficulty or not at all?

If it is not broad enough, then must it be made broad enough. It is unthinkable that we are to go on wasting and destroying the foundations of national prosperity simply because state and national authority alike are impotent to interfere for the protection of the public welfare.

It is well known that our present constitution was adopted after the experience of a decade had shown that the old Articles of Confederation by which the colonies were originally bound together were inadequate for the necessities of the nation. The constitution has transformed a mere aggregation of independent colonies, loosely held to one another, into a homogeneous nation. If the present constitution can be so construed as to meet urgent national necessities, well and good. Otherwise it must be altered to meet the requirements.

In saying this, moreover, we are in full accord with those who argue that the field of state action should not be unnecessarily interfered with or encroached upon. By all means let the several states do what they can; but a multitude of cases are arising where work done in one state affects people in another state. Cutting off the forests around the headwaters of the streams rising in the White Mountains sends a vast amount of silt down to fill up reservoirs in Massachusetts. Floods in the Delaware and Susquehanna rivers in Pennsylvania have their origin in tributaries which rise in New York. The proposed Southern Appalachian forest preserve would include lands in Maryland, Tennessee, Kentucky, Virginia, West Virginia, the Carolinas, Georgia and Alabama. The benefits from the preserve would be felt in all these states, and also in Pennsylvania and Ohio; but no one is wise enough to apportion the relative benefits to the several states.

These are but isolated examples of the practical difficulty in leaving to the states themselves the responsibility for protecting the public in this matter of the conservation of natural resources.

The same thing applies to other fields of activity as well as to forest preservation. New York City could obtain from the Housatonic in Connecticut an additional water supply at a saving of probably seventy-five million dollars compared with the cost of the supply from the Catskills. A recent Supreme Court decision prevents Staten Island from obtaining a water supply from the New Jersey mainland, from which it is separated only by a narrow channel, and compels it to obtain the supply from the distant mainland of New York State at enormous cost. The decision may be good law; but it would appear to lay heavy and unnecessary burdens upon the people affected.

It is a pleasure to note that the Public Service Commissions for both New York City and the remaining portions of New York State have adopted measures to secure uniform systems of accounts and statistics for not only the steam railways of the State but also the urban and interurban street railways and the gas and electric lighting corporations. It is a further cause for congratulation that the various corporations concerned appear to be willingly cooperating with the Commissions in formulating the necessary systems for uniform accounts and statistics. The task has been considerably lightened by the work previously done by the Interstate Commerce Commission and a number of the State railroad commissions, so far as steam railroads are concerned. Something has already been accomplished in the same way for street railways. In the gas and electric lighting fields the recent committee work of the American Gas Light Association (now the American Gas Institute) and the National Electric Light Association will of course be of great assistance. Every step towards an extension of uniformity in the statistics and accounts of the public service corporations of the country, whether national, State or local in character, is welcome and deserves the commendation and support of all interested in fairness in the matter of rates and service and progress in everything pertaining to efficiency and lowness of cost. We learn and are stimulated by studying the work of others, profiting often, at least potentially, as much by mistakes as by successes. But to profit in these ways, the results should be available in readily comparable form.

We are also pleased to note that the New York Public Service Commission for the Second

District (having jurisdiction over territory outside of New York City) has taken a strong stand as to including depreciation charges in the accounts and reports of railways and in including depreciation with the cost of railway service which must be met from earnings. It appears from the report of the Commission that in dealing with other parties interested in uniform accounts and statistics, a difference of opinion was found as to the desirability of deducting depreciation charges from revenue before determining the net earnings of the electric railways. The Commission therefore concluded to "advise all persons and corporations interested" that a depreciation account must be kept.

On the general subject of depreciation for railways the report of the Commission may be quoted as follows:

> After a road has once been constructed it rapidly depreciates in value. Large annual outlays are required to preserve the plant in working condition. This element of depreciation must be provided for out of the earnings. Unfortunately, however, this element has never been properly taken into consideration in the accounts of railway companies. It has been handled in various ways, not infrequently, we believe, by increasing the capitalization to provide new equipment as that in use wore out, and to make repairs to the road-bed, and in the replacing of rails, ties and other indispensable portions of the plant. The whole subject of depreciation accounts is in a confused and unsatisfactory condition. An effort is being made through a uniform system of accounts, which requires proper provision to be made for depreciation, to put it upon a better footing, and one which will enable those in authority to see what is justly and fairly the annual depreciation of the various parts of a railroad system and how much of the annual gross earnings should be set aside for that purpose.
>
> It should be constantly borne in mind that any reduction of rates which forbids a railroad from making proper provision for depreciation, is a serious and, indeed, irreparable injury to the public. It is essential to adequacy and efficiency of service that depreciation be provided for out of earnings, and earnings result from the rates charged. There is no escaping the proposition that the public must pay for the depreciation. Either the public must pay or the company itself must pay. The company itself has no means of paying except from the earnings, which of course means payment by the public, or by an increase of capitalization, and this is, as is apparent, a suicidal course and can only result in absolute financial chaos. No argument is required to show what it will lead to.

This is good sense, and in the long run the recognition and acceptance of the principles involved will be of benefit to all classes of public service corporations, as well as the public itself.

A correspondent sends us a county paper in Pennsylvania containing the following item describing one of the recent gasoline-electric motor cars:

> Last Saturday an electric car sped through Mt. Union on its way west making its own power as it went. There was no pulling and no pushing but a swift going west. The car had in it a dynamo which gets its power from the wheels of the car. This generated electricity which was used to make the wheels go round. In running down hill it packed the electric current into its chest to be used in going up the next hill. It had also a gasoline engine to start the car and its wheels going.

Evidently there are still many believers in the possibility of one's lifting himself by pulling on his boot straps.

NEEDED REFORMS IN THE COLLECTION AND DISPOSAL OF CITY REFUSE.

The unsatisfactory condition of garbage and refuse disposal in the United States has often been commented on in these columns. There has been some improvement of late, but it has not made itself strikingly evident in many cities as yet. Besides bad politics and general inefficiency in municipal administration, the chief hindrance to putting refuse collection and disposal on a satisfactory basis is the failure of the public and of non-technical city officials to recognize that the most difficult of the problems involved are engineering in character and will never be satisfactorily solved until they are entrusted to engineers. While the last statement refers primarily to the design and installation of garbage reduction and incineration works and the devising of systems of street cleaning and of refuse collection, it also applies in no small measure to the supervision of such works and systems after they are put in operation.

The idea that street cleaning and the collection and disposal of garbage and other city wastes are engineering problems, is slowly but surely gaining recognition in the country, as it has long been recognized abroad. Evidence of this is afforded by the more frequent employment of engineers to report to city authorities on such matters and by the occasional appointment of an engineer to the position of street cleaning commissioner.

A few months ago Mr. Foster Crowell, M. Am. Soc. C. E., was appointed as Street Cleaning Commissioner of New York City. Quite recently Mr. Guy C. Emerson, M. Am. Soc. C. E., was appointed as the head of the reorganized Street Department of Boston, in which capacity he will have general charge of street cleaning as well as practically all other matters relating to the streets excepting water-works distribution. In the way of special reports by engineers, one by Mr. Rudolph Hering, M. Am. Soc. C. E., on refuse collection and disposal at Milwaukee, had been before the public but a short time before there appeared a comprehensive report on the same general subject as related to New York City. The latter report was made by a commission of three engineers, including Mr. Hering and Messrs. H. de B. Parsons and Samuel Whinery, M.'s Am. Soc. C. E. The New York report is abstracted at length elsewhere in this issue. It may be noted that it goes extensively into street cleaning matters, as well as all other phases of city wastes disposal (excepting, of course, sewage), and although primarily local in character yet in the absence of a handbook on both street cleaning and refuse disposal this report will serve that purpose quite well. In this issue we also publish an abstract of a recent paper on refuse disposal, by Mr. Paul Hansen, one of the assistant engineers of the Ohio State Board of Health. In the course of his paper Mr. Hansen states that a somewhat extensive study of city refuse disposal is being made by the engineers of the Ohio State Board of Health.

This undertaking by the Ohio Board is the first instance that has come to our attention of a comprehensive study of the disposal of general city refuse by a State Board of Health. It is to be hoped that it will prove to be only a beginning of such investigations, for while the disposal of general city refuse is not so intimately related to the public health as are water supply and sewage disposal, yet they have most important bearings, at some points, upon health and disease, particularly as regards street cleaning and the prevention of street dust.

Both the report of the New York Commission and Mr. Hansen's paper lay special stress upon the need for more and better data regarding the nature and volume of city wastes and the cost of their removal and sanitary disposal. The New York Commission, it will be seen by our abstract, calls particular attention to the need for better accounting and cost-keeping on the part of the New York Street Cleaning Department. It also urges that a system of accounts and statistics uniform with those in other cities be adopted, in order that it may be possible to make comparative studies. It may be inferred from the remarks of the New York Commission that the New York reports are not uniform with themselves from year to year.* Unfortunately this is not peculiar to New York alone, and evidently reform in this particular should be attempted even if no effort is made to bring the New York statistics into line with those of other cities. In fact, before the latter could be done, it would be necessary to have a standard form of reporting street cleaning and refuse disposal statistics adopted by a number of cities, or at least formulated for their adoption. Perhaps this will yet be done by the United States Bureau of the Census in connection with its yearly "Statistics of Cities."

*We are pleased to note that new systems of accounting have recently been installed in the Department of Street Cleaning and in a number of other departments of New York City, as a result of cooperation between Mr. Herman A. Metz, Comptroller (280 Broadway), and the Bureau of Municipal Research (261 Broadway, New York City).

If needed comparable data as to street cleaning and refuse disposal were available, it would be far easier than it is now to determine in advance of actual trial what method of street cleaning or waste disposal is best suited to the local conditions of any municipality. In the absence of a large body of such data from numerous cities, such reports as that one relating to New York are particularly valuable, even though the numerous estimates that they include do emphasize the paucity of exact data. Still more valuable, because covering many cities of ordinary size instead of one large city which may be said to be composed of several municipalities of highly diversified character, promises to be the report on the Ohio investigations. It is to be hoped that there will be no lack of funds to carry on these investigations, for the results obtained may prove to be as valuable in their way as those following from the water and sewage purification investigations discussed in our issue of April 9, 1908.

It is interesting to note that both the New York report and Mr. Hansen's paper agree in giving garbage reduction a somewhat better status than has been the fashion in some quarters during the past few years. It was not so very long ago that Mr. Hansen joined in the authorship of a paper presented to the American Public Health Association, in the course of which garbage reduction was not put in a very favorable light (see foot-note to our abstract of Mr. Hansen's paper elsewhere in this issue). That the larger cities of the United States, or the contractors serving them, have generally adopted garbage reduction rather than incineration was shown in our issue of March 12, 1908.

There are indications in the New York report that the eminent engineers who formulated it would have preferred to recommend incineration for the whole city instead of a continuation of the present garbage reduction service for the greater part of the city and incinerating plants with heat utilization for two of the smaller outlying boroughs. Present conditions, however, did not seem to warrant the adoption of incinerating plants for the whole city. The fact is, that, although high-temperature refuse furnaces, with heat utilization, are generally accepted as the only proper thing throughout Great Britain and to a large extent on the continent of Europe, yet in this country the few and promising installations of this character are both too few and too recent to regard them as being beyond the experimental stage, American conditions considered.

It is well worth noting that in his appendix to the New York report, Mr. George A. Soper, M. Am. Soc. C. E., calls particular attention to the extent to which the refuse of European cities is disposed of on land, even where refuse destructors are in use. It is also to be noted that in Mr. Soper's opinion there is altogether too little known regarding the full cost of operating British refuse destructors. From such information as he obtained, however, he concluded that notwithstanding the revenue derived from the utilization of heat, clinker, etc., burning city refuse abroad is not so economical as has been generally considered, when the expenses for repairs, interest and depreciation on the plant were taken into account.

The inefficiency of ordinary street cleaning methods is well brought out by the New York report. Special study showed that the amount of dirt left on different classes of pavement by ordinary street cleaning operations ranges from 100 for sheet asphalt as a standard to 912 on granite blocks. It is for such reasons as this that there has been so much demand in the past for street sprinkling as a palliative when instead a really thorough cleaning should have been provided.

The lessons of the New York report, so far as street cleaning is concerned, are to abolish the sprinkling cart, which in some respects is more of a nuisance than the one it is designed to allay, and use the water so distributed, with as much more as may be necessary, to wash the street clean of dirt, thus making dust impossible until more dirt is deposited on the street.

Contrary to generally accepted ideas, the estimates of the New York Commission indicate, hand-hose washing, using specially designed nozzles, is cheaper than washing by the street flushing machines which have been introduced in a number of cities during the past few years, and which, it is claimed, clean streets much more cheaply, as they certainly do or should much more effectively, than is possible by either hand or machine sweeping. The estimates of the New York Commission on the cost of street cleaning by various methods promise to form one of the most valuable parts of its report. Since we reprint them elsewhere in this issue we need only say here that for washing by hand hose the Commission estimates that an outlay of only 25 cts. per 1,000 sq. yds. would be required against nearly three times as much for washing by means of the so-called machines.

The tax upon the municipal treasury caused by streets out of repair properly comes in for attention in the New York report. It is estimated that the cost of street cleaning in the boroughs of Manhattan, The Bronx and Brooklyn would be reduced by $375,000 a year if all the streets were in good repair. As our readers will understand, the added cost of street cleaning and in some cases the impossibility of adequate cleaning is only a beginning of the expense and damage that result from the miserable condition of street pavements in many of our cities.

In conclusion we would suggest that every engineer should do all in his power to bring home to city officials and taxpayers the crying need for a general reform in the street cleaning and refuse disposal methods in vogue in most of our cities. Better systems of statistics and accounts, uniformity of data as between different cities, so far as feasible, and entrusting the work to engineers, would result in greatly increased efficiency and decreased cost of service.

LETTERS TO THE EDITOR.

Air-Pressure Effects in the Battery Tunnel.

Sir: At a recent meeting of the Brooklyn Engineers' Club a paper upon the Battery tunnel from Manhattan to Brooklyn was ably presented. In the discussion that followed the paper Mr. Soper spoke, among other things, of the sensation felt in the ears and he accounted for it as due to change in elevation, as, for example, in going up and down elevators of the tall buildings in Manhattan.

It seems to me this sensation felt in the ears is easily explained when the tunnel and the means of its ventilation are considered. There is a ventilating shaft near Furman St. The speed of the train is about maximum when this point in the profile is reached, in passing it toward Manhattan, and the train strikes air that is almost at rest when compared to the train velocity. This produces a sudden and enormous concussion and greatly increased pressure of the air and we feel it very sharply in our ears. The shaft at the Battery end is so near the Bowling Green station that no such distinct sensation is felt when going across from Manhattan to Brooklyn yet the vertical fall and rise is the same.

Very truly yours,

J. Calvin Locke.

209 Montague St., Brooklyn, New York City, March 20, 1908.

The Permanency of Concrete.

Sir: There is being circulated through this part of our state by the clay-working interests, a lot of printed matter in which it is said that the life of cement is but from seven to fifteen years, and that if large tiles or sewers are constructed of concrete in proportions of 1 of cement and 4 of sand and gravel, the cement will all wash out in less than 15 years and leave only the gravel and sand. Have you ever had any experience in this line or are there any sewers in your state or any other state that have been in longer than 15 years?

Very truly yours,

Moeller Brothers.

Knierim, Iowa, April 11, 1908.

[Our judgment would be that the clay-working interests were throwing mud! It is not necessary to tell the readers of Engineering News that the cement does not "wash out of concrete," in fifteen years or any multiple of fifteen years. We publish the letter only as a curious illustration of the extremes to which misrepresentation is sometimes carried by rival business interests. It is the poorest sort of business policy to lie about a competitor, because when the lie is found out, as it is pretty certain to be, sooner or later, the reputation of the liar's goods, as well as that of the liar himself, is certain to suffer.—Ed.]

Another Reinforced-Concrete Bridge Designed with Dangerously High Working Stresses.

Sir: In Engineering News of Sept. 27, 1906, a reinforced-concrete bridge that had recently been erected was computed and a prediction made that it would collapse. In a later letter in that same issue, the collapse of the structure was reported. Attention was called at the same time to a 24-ft. span of similar construction that seemed to be too light for its specified loading. On April 6, 1908, this latter structure collapsed, and a view of the wreck is shown in the accompanying illustration. There was no load on the bridge at the time, and no evidence could be discovered of any undermining or weakening from recent floods. It appears simply to have settled down to a well-earned rest.

A REINFORCED-CONCRETE BRIDGE FAILURE.

Very truly yours,

Daniel B. Luten.

Pres. National Bridge Co.

Indianapolis, April 14, 1908.

[The bridge which failed was composed of reinforced-concrete girders carrying a floor slab. According to the statement made by our correspondent in our issue of Sept. 27, 1906, the bridge was built in July, 1906, and the centers were removed in August. An examination shortly after showed the center girder to be badly cracked.—Ed.]

Cracks in the Lower Flange of Reinforced-Concrete Beams.

Sir: Being much interested in reinforced concrete from the theoretical side, I find a difficulty in accepting some of the general assumptions as to stressing the steel and concrete. To start with, it is the usual custom in the case of a freely supported beam to take a working stress of about 15,000 lbs. per sq. in. in the steel, and to ignore the tensional strength of the concrete below the neutral axis. The modulus of elasticity of good concrete is about 2,000,000 lbs. per sq. in. and the ultimate tensile stress allowable without cracking is 200 lbs. per sq. in. Supposing therefore that the steel is stressed up to 15,000 lbs. and taking the modulus of elasticity of steel as 30,000,000 lbs. the extension of the steel will be:

$$\frac{15{,}000}{30{,}000{,}000} = \frac{1}{2{,}000} \text{ per unit length.}$$

On the other hand the ultimate allowable extension of the concrete will be $\frac{200}{2{,}000{,}000} = \frac{1}{10{,}000}$. The concrete will therefore rupture long before the steel is fully stressed. The ultimate strain on the concrete corresponds to a stress of 3,000 lbs. per sq. in. on the steel which is uneconomical. The concrete will therefore have a multitude of fine cracks immediately round the bar. These cracks must surely ultimately form a source of danger and it does not appear to me reasonable to suppose that engineers will be satisfied with beams and slabs which are probably ruptured once the test load has been applied.

I shall be extremely glad if you will allow this letter to appear in your columns and invite replies thereon.

Yours faithfully,

G. S. Coleman.

Engineers' Dept., City Surveyor's Office, Manchester, Eng., April 4, 1908.

[Tests on reinforced-concrete beams have demonstrated the occurrence of such cracks as are noted by our correspondent, but it is probable that he has set the stress at which they occur slightly low, as the latest and best experiments show the first cracking in the concrete at about 5,000 lbs. per sq. ft. in the steel. These tension cracks have been the subject of considerable dispute, but the general opinion now seems to be that they are of minor importance in the safety of the beam. The cracks can never extend above the neutral axis, nor otherwise become so great as to destroy the ability of the steel to resist tension or the concrete in the upper flange to resist compression. The one danger that they do present is the opportunity they allow for corroding influences to penetrate to the steel and eventually to destroy it. However, the cracks are so minute under working loads that the probability of such corroding is remote.

In regard to these cracks it is interesting to note that they are so small that M. Considere, in his first tests on reinforced-concrete beams, concluded they did not exist, and to explain their absence, promulgated the theory that concrete reinforced with metal had a much greater elasticity than plain concrete. Prof. Turneaure of the University of Wisconsin disproved this by soaking test beams in water, thereby making apparent by moist spots the first indication of concrete cracking. By this method it was shown that the concrete with embedded steel began to fail in tension at about the same or a slightly higher value than the plain material.—Ed.]

"Why Should the Profession of Civil Engineering Appeal to a College Man?"

Sir: A very important significance attaches to the query addressed by some students of Yale to Mr. Stevens, the answer to which appears in Engineering News of April 2.

The people of the country see the value of a technical education and are striving to equip their sons as thoroughly as possible for the life before them when they encourage the study of engineering. It is becoming absolutely necessary in these days of high industrial development, that the leading men in great undertakings should have a good technical training. Much of the incompetency which now exists in the management of great corporations is due to lack of this training. Politicians, statesmen and financiers must have engineering knowledge. The great men of the country must be engineers.

The reason that engineers have not as yet become more prominent in public affairs is because in most cases their education and subsequent training has been too narrow.

Our great men must have a knowledge of the humanities, of law and of literature and be able to talk and write both fluently and convincingly; but in addition to this they must be able to understand clearly the technical side of every great movement for the physical betterment of the country.

Having awakened very tardily to this fact, the educators are now busily engaged in seeking some means to induce the young men who study engineering to also study the subjects which have been taught exclusively in the arts and classical courses. A recent press comment from Europe upon Cornell University is as follows:

> The sharpest impression one carries away from it is a picture of a highly developed technical school, with a College of Arts as appendix, which might almost be removed without serious danger to the body to which it belongs.

With a new Hall of Humanities, not needed for the students of arts, they are seeking to strengthen the arts course by offering to the engineering students a five-year course which will include much of the work of the arts course.

A student is fortunate indeed if in addition to his technical studies, he can find time and money to prepare himself for a broader and a higher field than that which pertains strictly to the design and construction of engineering works. The profession has suffered very greatly through the lack of able representatives and advocates who could and would present correctly to the public the professional side of many great undertakings. They have suffered themselves to be misrepresented by newspaper men who, however brilliant, have never yet been able to grasp the technical side of an engineering subject. With a broader education and a growing interest in public matters we may expect the next generation of engineers to be most conspicuous in national affairs.

Albert J. Himes.

10,528 Bryant Ave., N. E., Cleveland, Ohio, April 6, 1908.

The Use of the Stadia in Railway Location.

Sir: The letter of Mr. John K. Flick, in your issue of March 26, 1908, is interesting but hardly convincing; at least, that part of it which refers particularly to the conduct of railroad location surveys in the tropics.

Mr. Flick asks, "Who ever heard of a stadia survey being the most rapid and consequently the most economical in the jungles of South America?"

It might be asked in return: Why is a rapid survey "consequently" the most economical? Under some conditions, and with many and various qualifications, it might, or might not be; the writer, however, has personally made surveys in the "jungles" of tropical South America which were both rapid and economical and, in his opinion, more so than is possible by the use of any other method, giving equal results and information of the kind absolutely necessary for the projection of the final route to be surveyed in detail.

Apparently Mr. Flick imagines that the use of the stadia is confined to topographical surveys of areas. As a matter of fact, however, as is well known to those familiar with its use, it has many other uses, especially for reconnaissance and exploratory surveys for railroads, whether in timber or open country.

The great difficulty in making surveys for a railroad location in timbered country anywhere, and especially so where the undergrowth is as dense as it is in some parts of the tropics, and where there are either no maps at all or maps which are worse than useless owing to their incorrectness, is to get an adequate idea of the general topography of the country. Practically the only means available in such a case is the running of a number of exploratory lines, more or less parallel to each other in the general direction of the routes to be explored, the number of lines and their distance apart being determined by local circumstances.

If the underbrush is very dense, lines have to be cut through it by main strength. The writer has seen many miles where with a force of ten or twelve *macheteros* (axemen), all of them picked men, and wonderfully skilled in the use of their universal tool, the machete, it has been impossible to cut more than about a kilometer (½ mile) a day, and then the narrow *trocha* (cut) was hardly more than a tunnel through the living vegetation just large enough to walk through. It was practically impossible to leave the line in any direction even with a couple of men to clear a way, or even to see more than a yard or two on either side of the line. This is an extreme case, of course, but on the line which the writer has in mind, where he filled a subordinate position, an average of about a mile a day of preliminary line was all that could be accomplished for several months by the hardest physical work.

The organization on this work consisted of six Americans and fifteen natives in the field, and a cook and two men in camp, the field party being organized as follows: Chief of party, transitman, 2 chainmen, leveller, rodman, 15 natives. The line run was only preliminary, and practically all the information obtained was an angle line giving distance and direction, a profile of this line, and the most meager notes of the topography in the immediate vicinity of the line run. Of the 15 natives, one was assigned to each instrumentman to carry the instrument, one cut and drove stakes, one held the back flag, leaving only 11 to cut. Each of the Americans was provided with a horse which was often left in camp during the day.

The information could have been obtained by stadia methods in the same length of time and at much less than half the expense. No more clearing would have been required to see the stadia rod at any point along the line than was required to see and measure the transit line or to see the level rod, and one instrumentman could have done all the work with one other engineer to direct the cutting. In fact, on a pinch, the writer has made surveys of this kind alone.

The fact that the stadia may be, and has been abused by its use for work for which it is not well adapted, is, of course, no argument with those who have common sense and intelligence; its use is limited to exploration surveys preliminary to the selection of a route to be surveyed in detail, and there is no question that, bearing in mind its limitations, it has a much wider field of use on railroad location, *especially in the jungles of South America*, than it has been hitherto applied to.

The writer's surmise, expressed in his letter in your issue of November 7, 1907, that the number of men taken down to Brazil for the work on the Madeira & Mamoré R. R. seemed to be unnecessarily large for preliminary work, seems to be confirmed by Mr. Flick's statement in the fourth paragraph of his letter, but it is necessary to have a much clearer and comprehensive statement of the work actually accomplished before any explicit criticisms of the work can be made, other than those based on the original article in your issue of Oct. 24, 1907, by Mr. Liebel.

Yours very truly,

F. Lavis, M. Am. Soc. C. E.

134 Elm Ave., Mt. Vernon, N. Y., April 6, 1908.

Laying Out Curved Boundaries for City Lots.

Sir: In looking over the article for "Laying Out Curved Boundaries for City Lots" submitted by "H. H. T." in Engineering News, March 12, 1908, p. 285, I find that I cannot agree with him. The "Deflection Angles for Curves" is figured for a value $c = \frac{R}{100}$, and the unit angle is determined by the formula

$$c = 2R \sin \tfrac{1}{2} \triangle = 2R \sin \delta,$$

or,

$$\sin \delta = \frac{c}{2R} = \frac{1}{200} \text{ (where } R=100c\text{)}; \quad \frac{2}{200} \text{ (where } R=50c\text{)};$$

etc.

It is misleading and in error to say, for instance, that the total deflection angle for 20 stations of chord length $\frac{R}{100}$ is the same as for 5 stations of chord length $\frac{R}{25}$. The *unit* angles δ in the following table are for the several assumed chord lengths, and, in actual work of staking out, as in railroad curves, we must use the unit angle that corresponds to the desired chord length, not any other.

DEFLECTION ANGLES FOR CURVES.
Constant for all radii.

R	δ	R	δ
100c	0°-17′-11.33″	6¼c	4°-35′-18.84″
50c	0°-34′-22.68″	$5^{15}/_{17}$c	4°-52′-33.69″
33⅓c	0°-51′-34.09″	$5^{5}/_{9}$c	5°-09′-48.99″
25c	1°-08′-45.57″	$5^{5}/_{19}$c	5°-27′-04.75″
20c	1°-25′-57.16″	5c	5°-44′-21.02″
16⅔c	1°-43′-08.87″	$4^{18}/_{21}$c	6°-01′-37.80″
$14^{2}/_{7}$c	2°-00′-20.74″	$4^{8}/_{11}$c	6°-18′-55.14″
$12^{1}/_{5}$c	2°-17′-32.79″	$4^{8}/_{23}$c	6°-36′-13.05″
$11^{1}/_{9}$c	2°-34′-45.05″	$4^{1}/_{6}$c	6°-53′-31.57″
10c	2°-51′-57.54″	4c	7°-10′-50.72″
$9^{1}/_{11}$c	3°-09′-10.29″	$3^{11}/_{13}$c	7°-28′-10.54″
$8^{2}/_{3}$c	3°-26′-23.33″	$3^{19}/_{27}$c	7°-45′-31.04″
$7^{9}/_{13}$c	3°-43′-36.67″	$3^{4}/_{7}$c	8°-02′-52.24″
$7^{1}/_{7}$c	4°-00′-50.35″	$3^{13}/_{29}$c	8°-20′-14.20″
$6^{23}/_{29}$c	4°-18′-04.40″	$3^{1}/_{3}$c	8°-37′-36.95″

Very truly yours,

J. Calvin Locke.

209 Montague St., Brooklyn, N. Y., March 28, 1908.

Piece-Work in a Bridge Shop.

Sir: Referring to a query in your issue of March 26, I wish to submit a scheme by which I had shop work on bridges successfully done on the piece-work plan during three years that I acted as manager of a small bridge shop.

Following the example of another shop, we started by trying to let separate contracts to individual workmen for different operations such as punching, reaming, riveting, etc. A card was issued on which the number of holes, rivets, etc., and the price per item and in total were recorded. The men worked by the hour, and were paid each Saturday for the number of hours. After each month, statements of piece-work finished in the preceding month were made up, and on each card was recorded the number of hours worked on the job and the wages paid for same, and their total compared with the amount of the contract. On the first Saturday of each month these cards were returned to the workmen, and, if the work had been done in less time than provided for, the balance was then paid out. If the actual number of hours worked exceeded that of the contract, the shortage was not subtracted from the week's wages, but put on account of the workman to be settled with future balances in his favor, it being the policy of the firm to guarantee the wages per hour.

This plan was not successful, chiefly owing to the following reasons:

Part of the work in the shop, such as marking of holes, assembling, etc., remained to be done by the hour; this part became more expensive, and it proved difficult to ascertain what numbers of hours had been put in on piece work, if there were many small jobs in the shop. As this work was done by the most skilled men they complained that they could not make as much money as less skilled workmen, who as a rule did the punching, riveting, etc., on the piece-work plan, and these latter workmen had few inducements to do their work carefully. Furthermore, the scale of prices, and its changes (produced by newer equipment and methods) caused much trouble and discontent. In general the plan did not secure proper reward to the men who put in their best efforts, and the saving was not in proportion to the accounting necessary.

This plan was abandoned and we introduced another one as follows: Each job was divided into parts which could about be finished in a month by the gang to be employed on it, each part as a rule amounting to something like $100 or $200. Each part included all the work to be done on a definite part of a structure or a group of members. For each of these parts or contracts an estimate of the labor cost was made:

Straightening of plates and shapes, per 100 lbs.; Making of templets (to be estimated with regard to numbers of holes and number of pieces); Marking of material (to be estimated with regard to number of pieces); Marking of holes, per 100 (scale for a few different kinds and sizes of work); Shearing plates, per lineal foot (following a scale for different thicknesses); Cutting shapes into lengths, per 100 (scale for different groups of shapes and for square and skew cuts); Grinding of edges, per lineal foot for plates, per 100 for shapes; Punching of holes, per 100 (scale for punching-machine to be used, for tie plates, for long members punched over the full length or only partially so); Assembling of members (estimated); Reaming, per 100 (scale for different sizes of holes and different kinds of work); Riveting (scale for different sizes of rivets, different tools to be used and sort of work); Calking of rivets, per 100 (scale for different sizes); Forgings, in most cases to be estimated; Special work, not mentioned above, to be estimated; Painting and marking of members, to be estimated; Handling of material and members, per 100 lbs.

Milling of surfaces and boring of pin holes were not included in the piece-work, owing to the fact that the shop in question did much repairing of machinery, but this might easily be done.

The estimate being made, a card was issued which gave a clear specification of the work to be done, and mentioned only the lump sum. It was left to the foremen to divide the force in different gangs for the different jobs, and to shift them if necessary, this last, however, to be avoided as much as possible. After the job was done the cards or forms were returned to the office and on each was recorded the total number of hours worked on it by each man and the wages paid. Surplus or shortage were divided in the same proportion as the men had participated in the total of actually paid wages. The forms showing these items were returned to the workmen. On the first Saturday of each month the balance of the different jobs was paid out.

This scheme proved to be satisfactory to both the management and the workmen. It was a guarantee to the former that the labor cost of a contract would be kept below the estimate if possible, and it offered an opportunity to the latter of earning a surplus on their wages varying from 10% to 20%. It proved not to be difficult to adjust scales, which fixed the greater part of the work by far. The operations which had to be estimated formed a minor part of the total and this estimating could be done closely by comparing former estimates.

A few additional attractive features of the scheme were: the men individually tried constantly to facilitate their operations by small tools, fixtures and means, and complained to the management if anything interfered with their doing their work efficiently; at the same time favoring themselves by discharge of slow men. The management in preparing bids no longer relied on estimates of labor cost per 100 lbs., but had estimates of same prepared on the plan as explained, which were much closer and proved a great help to make successful bids.

It might appear as if the preparing of these estimates might require much time; it proved, however, that with some training they could be made quite rapidly, the drawings furnishing most of the necessary data.

I would not pretend that under this scheme there never was any trouble. This, however, might under no system be wholly avoided in this difficult matter. In general I believe that a scheme such as this will prove a help to the management in keeping down manufactur-

ing cost, and an opportunity to the workmen to increase their wages by taking interest in their work.

Very truly yours,

F. Ensink.

Bridge Dept., Northern Pacific Ry., St. Paul, Minn., April 4, 1908.

Calculating Net Section of Riveted Tension Members and Fixing Rivet Stagger.

Sir: The requirements of specifications in common use in regard to determining the net section of riveted tension members are of two general types. In one the area of the hole A (Fig. 1) would have to be deducted from the area of the transverse section BC in case the spacing p were less than a certain fixed distance; the other kind of specification would require the hole A to be deducted if the net section along AB and AC were less than a certain percentage in excess of the net section along BC. Either method leads to an absurd result in the case of large value of w. For instance, if $\frac{w}{2}$ is 20 ins., a certain well-known specification of the first type would require p to be not less than 2½ ins., while a widely-used specification of the second kind would require p to be not less than 17 ins. Evidently the true value of p is somewhere between these extremes.

Fig. 1

The following method is submitted as being more rational and at the same time quite easy of application:

Let d = diameter of hole (assumed 1 in. in the case of ⅞-in. rivets).

ϕ = angle ABC.

s = tensile unit-stress along the section BC.

s_n = normal unit-stress along the section BA.

s_t = shearing unit-stress along the section BA.

s_m = maximum tensile unit-stress due to s_n and s_t.

p = the required minimum longitudinal spacing corresponding to the transverse spacing w.

The total stress on BD or CD is $\frac{w-d}{2}\,s$.

Then $$s_n = \frac{\frac{w-d}{2}\,s\cos\phi}{l-d} = \frac{s\,w\,(w-d)}{4\,l\,(l-d)},$$

and $$s_t = s_n \tan\phi = \frac{s\,p\,(w-d)}{2\,l\,(l-d)},$$

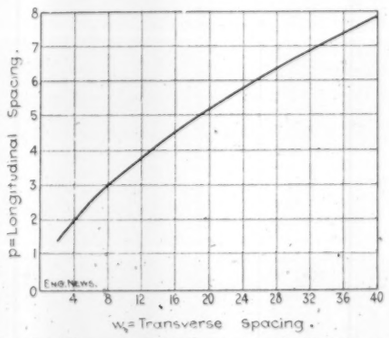

Fig. 2. Diagram of Minimum Longitudinal Spacing of Rivets if Alternate Holes are not Deducted.

For 1-in. holes for ⅞-in. rivets:

Count out any hole (as A) between B and C if the distance p is less than that given by the diagram.

The maximum tensile unit-stress due to s_n and s_t is given by the well-known formula:

$$s_m = \frac{s_n}{2} + \sqrt{\left(\frac{s_n}{2}\right)^2 + s_t^2}$$

Substituting the above values of s_n and s_t,

$$s_m = \frac{s\,w\,(w-d)}{8\,l\,(l-d)} + \sqrt{\left\{\frac{s\,w\,(w-d)}{8\,l\,(l-d)}\right\}^2 + \left\{\frac{s\,p\,(w-d)}{2\,l\,(l-d)}\right\}^2}$$

$$= \frac{s\,(w-d)}{8\,l\,(l-d)}\left(w + \sqrt{w^2+16\,p^2}\right)$$

But

$$l = \sqrt{\frac{w^2}{4} + p^2} = \frac{1}{2}\sqrt{w^2+4\,p^2}$$

Hence

$$S_m = \frac{s\,(w-d)\,(w+\sqrt{w^2+16\,p^2})}{2\,w^2+8\,p^2-4\,d\,\sqrt{w^2+4\,p^2}}$$

Now s_m should not exceed s. Putting $s_m = s$ we get the equation

$$(w-d)(w+\sqrt{w^2+16\,p^2}) = 2\,w^2+8\,p^2-4\,d\,\sqrt{w^2+4p^2}$$

or

$$w^2+8\,p^2+dw = (w-d)\,\sqrt{w^2+16\,p^2}+4\,d\,\sqrt{w^2+4\,p^2}$$

By assuming values of w and d we can find the corresponding values of p by trial. When plotted to scale these form a curve like that shown, which is for 1 in. holes. The hole A was assumed midway between B and C, but in case it is not it will be sufficient for the purpose to deduct hole A if the distance AD is less than the distance p corresponding to $w = BC$.

Yours truly, Victor H. Cochrane.

New Nelson Bldg., Kansas City, Mo., Feb. 27, 1908.

More Concerning Answers to Applicants for Positions.

Sir: In Engineering News of April 2, "A Victim" makes bitter complaint because his application for a position, in response to an ad., went unanswered, and his stamp was appropriated.

The remedy is simple: In answering a "blind" ad., don't enclose a stamp; if an advertisement for help appears over the name of a firm, don't enclose a stamp. In either of the above cases, if the employer wants you, you will hear from him; if he doesn't, you are the gainer by two cents.

The man who uses a blind ad. does so, in many cases, to save himself from the pain of turning down a couple of million personal applications from correspondence school graduates (no boquets please—I am an "I. C. S." myself), students from manual training schools, etc., who, while willing enough, cannot at once take the place of an experienced engineer or chief draftsman.

I secured one position by answering an advertisement in Engineering News, and have had other offers from the same source. I never enclose a stamp in such cases.

In making an unsolicited application, however, a stamp should always be enclosed. In 95% of the cases where this is done, an answer may confidently be expected. In the other 5%, I don't care to express my opinion of the employer (I may want a job with him some time), but in times like the present there is some excuse even for him.

H. M. K.

Joliet, Ill., April 5, 1908.

[There are many other good reasons besides that mentioned above why a responsible firm or company when advertising for men prefers not to do so over its own name. In fact it is the rule that a concern advertising for help does not wish the fact known to its competitors or to its present staff.—Ed.]

More Concerning "The Function of the Consulting Engineer" and Bridge Companies.

Sir: I would add to Mr. J. A. L. Waddell's interesting letter in your issue of Feb. 13, 1908, what no one will deny, that that engineer should be chosen for bridge design who is most expert by reason of training and experience, for the particular problem in hand. When the location and structure are such that an engineer of Mr. Waddell's attainments can be secured, very good; but when such experts are to be found in the employ of bridge companies it would be folly to reject them for engineers of less experience simply because the latter choose to call themselves consulting engineers.

Mr. Waddell implies that the designing of a bridge by the constructor himself may lead to dishonest practices; and a popular impression prevails that to permit a bridge contractor to bid upon his own plans is placing a premium on dishonesty. Yet the very reverse is much nearer the truth. What method of design and construction could be better calculated to breed dishonesty among contractors than to require them to construct a structure on plans prepared by an engineer and in which the constructor has no interest save to place the materials at least cost. While on the other hand, what constructor would not take pride in executing work designed by himself and in which the whole undivided responsibility for a satisfactory structure rests upon his own shoulders? He could most assuredly not be asked to guarantee the structure built according to the former method, while he could hardly find reasonable grounds for refusing to guarantee the structure built under the latter method.

On plans prepared by a consulting engineer and executed by an independent contractor, the contractor gives bond to provide satisfactory materials as specified. The engineer gives no bond and in case of failure of the structure is irresponsible. Granting that the services of experts may be secured either from consulting engineers or from constructors, that system would seem to be best which places design, construction and responsibility upon the constructor subject to the approval of the consulting engineer, who should be what the name implies, a consulting engineer and not a designing engineer.

Particular instances are not conclusive. It would be possible to cite hundreds of cases where purchasers have suffered loss by relying on consulting engineers instead of trusting to the constructors themselves, for their designs. But the lesson is driven home very forcibly in the Quebec Bridge disaster; would not the purchaser of that structure have been in a much stronger position financially had he not given to one of the most eminent consulting engineers in that line, the authority to restrict and modify the design to such an extent that it is now perhaps difficult to say whether or not the constructor must make good the loss?

Very truly yours, Daniel B. Luten,
President National Bridge Co.

Indianapolis, Ind., March 14, 1908.

Sir: In your issue dated Feb. 13, 1908, Mr. J. A. L. Waddell writes as follows:

From page 475 in your issue of Oct. 31, I quote the following:

> Q. Do you consider that it is wise practice when building a bridge of novel character and unprecedented dimensions to place the design of the structure and of the methods of erection in the hands of the mechanically trained staff of a contracting company, and if not, why was this practice allowed in this case?
>
> A. In answer to this question, it is the general practice in America to have the mechanically trained staff of contracting companies prepare the working plans. As a rule, no engineer could afford to maintain a staff of such character and no corporation would listen to a fee that would cover any such expense.

And from your editorial on page 587 in the issue of Nov. 28 I quote this statement:

> The normal and proper way of constructing any great engineering work which is done by contract is for two engineering organizations to work together. The one representing the contractor and the other representing the purchaser can check each other's errors, can study the work from different points of view and can produce a better and more economical result working in cooperation than either could working alone.

If these statements are not contradicted, the general public will be led to believe that American bridge specialists are incapable of designing great bridges, that they have to fall back upon the superior (?) knowledge of the bridge companies, and that their sole function is to check the numerical work (and perhaps also the honesty) of the manufacturer's designers.

I have read this last paragraph of Mr. Waddell's with great care and I am wholly unable to find anything in the quotations cited to justify his charges. Mr. Cooper's answer to the question of the Quebec Bridge Commission and your editorial do not reflect in any way upon American bridge specialists. Nor do I find any warrant for his slur upon the honesty of the manufacturers' designers. Where is there in the quotations any suggestion that the knowledge of the bridge companies is either inferior or superior to that of other experts? Mr. Waddell would have us believe that consulting engineers have nothing whatever to learn from bridge manufacturers and that a check by one on the work of the other cannot produce beneficient results. There is an old saying, "Two heads are better than one, even if one is a sheep's head."

Speaking of the economy gained by allowing bridge companies to prepare working drawings, Mr. Waddell says:

> Such economy as the practice entails springs from the fact that the contractor employs cheaper and less competent men to do the work than the consulting engineer does.

I am at a loss to understand why this should be assumed by Mr. Waddell to be the case. My experience with most consulting engineers who attempt to design bridges is that they have not the amount of business to warrant their employing high-salaried men, whereas the reverse is the case with manufacturing concerns. Note also that Mr. Cooper, quite as eminent a consulting engineer on bridges as Mr. Waddell, holds the very opposite opinion, as shown in his answer quoted above.

I am curious to know where consulting engineers gain the experience in bridge construction that makes them in Mr. Waddell's opinion so far superior to manufacturers as designers. From technical schools perhaps, trained by instructors who never had to do with practical construction, and possibly from inspecting erection by bridge experts. It has long been supposed that nothing could entirely take the place of experience, in other lines of industry, and I do not see why the bridge business should be made an exception. Mr. Waddell is perhaps making the mistake of trying to compare a good consulting engineer with a poor manufacturer. If it is so desirable that consulting engineers should prepare the plans for bridges, why is it not desirable to extend this practice to other lines of manufacture; why not buy our desks and tables in the same way? Why not employ engineers to design wagons, locomotives, automobiles, and then let the contract to the lowest bidder on the engineer's plans? Such a procedure would be absurd of course. Why is it not then absurd when applied to bridges?

If Mr. Waddell would assume the broad viewpoint of Mr. Cooper and realize that what is best for society is best for the engineer, his business as consulting engineer would outgrow the feature of designing.

Mr. Waddell concludes:

> A true bridge expert is an engineer who is thoroughly posted in every detail of designing and construction,

and who has had ample experience not only in the designing office, but also in the rolling mills, the bridge shops, the testing room, and in the field.

In the offices of how many consulting engineers can such men be found? And where is the bridge company that does not employ them?

Yours respectfully,

H. C. Malott.

San Francisco, March 23, 1908.

The Field of the Fruhling Dredge.

Sir: By permission of Colonel W. L. Marshall, U. S. Engineer Officer in charge of New York Harbor Improvement, I send you abstracts from records of the government dredge "Gedney," for comparison with work of the Fruhling dredge, in Mr. Reid's very interesting paper in your issue of March 5, 1908, Table II.

The "Gedney," now used for maintenance of "old channels" in lower bay, New York Harbor, is a wooden, twin-screw ship, built in 1887; 157 ft. × 37 ft. × 16 ft., with 16.5 ft. light draft; it has a carrying capacity under favorable weather conditions of 735 cu. yds. of sand; is equipped with two 15-in. flexible trailing suction pipes with "hook-ended" mouthpiece 3 ft. 10 ins. wide provided with grids giving water-ways, 4 × 6 ins. Each suction is served by a vertical centrifugal pump with shrouded impeller 3 ft., 3.5 ins. in diameter, 17 ins. wide at the periphery, belt connected to a 15 × 20-in. horizontal engine, developing approximately 100 I. HP. The ship is manned by a single crew.

In pumping sand a speed of 225 r. p. m. of pumps is maintained, producing 26 to 27 ins. vacuum in suction about 4 ft. from entrance to pump; this speed is reduced to a minimum of 210 r. p. m. when dredging in mud. In pumping clayey mud, the material frequently comes from the discharge pipe in chunks roughly 9 ins. in diameter, as if chopped from a plastic mass by each vane of impeller; these chunks will retain their shape after long continued sun drying; the chunks are accompanied by a continuous flow of mud of a consistency varying from that of thin grout to that of thick mortar. A very large proportion of the mud raised, goes overboard with the excess water, is carried away by the tide, and settles on the adjacent shoals, but as the object of the dredging is to excavate a channel this is not an unmixed evil, except of course from the view-point of a contractor who is paid by hopper measurement. The pumping is continued with due regard to the weather conditions and the day's work, until the draft of the ship is considered sufficient. The load is measured in the hoppers as soon as the pumps are stopped, and while running to dump the mud will settle, particularly in a sea-way, two feet or more; a shrinkage of 18% of load has been observed. In pumping sand, the material leaves the discharge as a quick-sand, and the coarser and heavier the sand the less is there carried overboard; within a minute or two after the pumps are stopped one can walk all over the sand in the hoppers.

It would be of interest to know the depth at which the trials of the "Fruhling VII." were made; this question of depth is very important. I have raised as an average of a day's work 24.8 cu. yds. of sand per minute from a depth of 18 to 20 ft., while under approximately similar conditions, in from 28 to 35 ft., 17 cu. yds. was the best I could obtain. A detailed description of the Fruhling pump would also be interesting; probably it has much to do with the results reported.

The depth of cut made by drags has been observed by direct measurement in clear water where soundings in the ditch and on virgin ground, alongside were easily made in 20 ft. of water while dredging in sand, and a number of these soundings showed 4 ins. of depth; taking average speed of ship and width of scrapers on drags, this figured out at 14 cu. yds. of excavation per minute, while the hopper measurements for the load showed 17 cu. yds. per minute retained. From Mr. Reid's minimum figures, I compute 155 cu. yds. per minute excavation with 33 cu. yds. retained.

Very respectfully. Dunbar F. Haasis,
Supt. U. S. E. Dept.

Perth Amboy, N. J., March 28, 1908.

In our issue of March 19, Mr. H. S. Wood, M. Am. Soc. C. E., made several queries concerning the field of the Fruhling system of suction dredging as described by Mr. John Reid in our issue of March 5. Mr. Reid has replied to Mr. Wood and to Mr. Haasis in the following letter.

Sir: I regret that, owing to absence in the South, my attention has only now been called to Mr. H. S. Wood's letter on the Fruhling Dredge in your issue of March 21. In reply I would say that while it is not always gratifying to see ourselves "as ithers see us" the experience does not lack interest or instruction even if the critic's views are somewhat tinged with what the late Lord Kelvin was wont to term the "bias of preconceived notions." At first it might appear as if Mr. Wood had almost become a believer in the utility of the Fruhling dredge but, on further consideration, it is evident that his desire is rather by the faintness of his praise to condemn the hopper suction-dredge of every size and type.

That your correspondent is a very able and enthusiastic exponent of the stationary, cutter-head suction-dredge goes without saying, but in the present case his enthusiasm has apparently somewhat blinded his perception. In no part of my article on the Fruhling dredge did I deal with it otherwise than as a self-contained, hopper, suction dredge nor did I claim for it in any way any of the scope or functions of the cutter-head type of stationary dredge. It is true that I referred to the use of a short pipe line for the alternative discharge of the hopper contents when dumping through bottom doors cannot be arranged for and also to the incidental advantage that in such a method of discharge the material being brought near to fill would be pumped ashore in a more solid form than is now usual. My comparisons, however, are exclusively with other well known forms of hopper suction-dredges and my claims for the Fruhling system were based solely on its use as such. On the other hand Mr. Wood has discussed my statements almost entirely from the point of view of a designer and user of the cutter head suction dredge and on that account his queries and criticisms seem to be in many cases wide of the mark. Furthermore he misquotes me as saying that the Fruhling dredge will "revolutionize the suction dredge," whereas what I did say was that it will "*revolutionize present practice in submarine dredging in soft or free materials such as sand or mud.*" The words in italics were deliberately chosen by me to indicate the exact scope of the work for the Fruhling dredge and they surely expressly exclude work in the cutting of self-flotation through hard material or marshy country or low upland, which is obviously a special field of the cutter dredge.

Under these circumstances to reply categorically to the queries raised by Mr. Wood would only result in further misunderstanding but a short comment on his queries and criticisms may present some further information of interest to your readers. It appears desirable to reprint each of Mr. Wood's questions before my answer, for clearness.

Q. How can a Fruhling dredge cut its own way through shallow water, marsh or low upland?

A. The Fruhling dredge as described in my article is a sea going vessel with large propelling power. The type shown is no more suited to cross-country navigation than is an Atlantic liner and there is no desire or intention to employ it in such work.

Q. How can it clean up along the sides and corners of slips and bulkheads?

A. It is unfortunate that your correspondent should first ask this question, apparently for the sake of information, but that further on in his letter without waiting for our reply, he should decide right off that such work is beyond our scope. Nothing is easier than to theorize about what is unfamiliar, but the conclusions thus arrived at are apt to be misleading. If Mr. Wood will follow the example of the many engineers, Americans and others, who have inspected the work of the little dredge "Nickolaus," for the last six or seven years employed at the Elbe entrance of the Kiel canal, he will find a Fruhling hopper suction dredge cleaning up alongside dock quays and lock sills with complete success. I most certainly claim this line of work as one for which the Fruhling dredge is specially adapted. One thing is certain, as no suction forces extend beyond the dredge head, it is not possible for it to accidentally undermine the quay walls and bulkheads alongside of which it is operating. That serious damage has been done to loaded quays and decks by just such accidental dredge work with ordinary suction dredges is very well known.

Q. What would happen to a Fruhling drag-excavator when passing over an old wreck, a rock or a heavy lost anchor?

A. It will be admitted that old wrecks, rocks, lost anchors and so forth are scarcely the material which a suction dredge of any kind should be called upon to pass and it would scarcely be counted to a dredge-master's credit that he sounded for such material with so expensive an outfit as a first-class hopper-dredge. However, assuming that it might be possible to bring up this Fruhling dredge head "all standing" on an old wreck or such like, the resulting damage would be practically nil. At the worst the heavy portable cast steel teeth attached to the cutting edge of the head would be detached. As they are bolted on exactly as on a steam-shovel bucket the expenditure of time and money on repairs would be trifling. Can Mr. Wood claim a similar result for his cutter head? Is it not the case that an old wreck would leave his cutter minus its blades or minus its spur wheel teeth and that there would be a heavy bill for repairs and loss of time? The Fruhling dredge head mouth being protected as stated in my article, by a grid of steel baffle plates, does not concern itself over stones, anchors, logs and so forth which might strip a cutter head down to the bare hub in spite of the safety releasing gear on the cutter driving engine.

Q. Is it pretended that the thick material handled by the Fruhling dredge could be commercially pumped 11,000 ft. with a 17-ft. lift as a dredge of the North American Dredging Company has recently done at Galveston?

A. If Mr. Wood had revealed the fact that his dredge was assisted in this tour-de-force by two repumping or boosting plants of great power in his 11,000 ft. of pipe line he would have lost none of the credit which he evidently desires for his company. As we can hardly assume that he has forgotten this fact the question seems somewhat disingenuous. As no Fruhling hopper dredge has ever been employed on such a proposition, or is ever likely to be, it will scarcely serve any useful purpose to discuss what the fluidity of discharged material would be at pipe line end.

Q. Is it claimed that this dredge will leave a more even bottom than an ordinary suction dredge radially sweeping its cutter about a vertical spud and having its cutter head built as a conical frustum to cut a level bottom?

A. Such is certainly my claim for the Fruhling dredge which operates by making level and accurate cuts to an absolutely defined depth and cannot operate otherwise. That a cutter-head dredge does anything of the kind in soft material I can scarcely admit. No doubt in theory the cone shaped head swung radially round a stern spud is a beautiful idea, but in sand or mud the more usual plan of operation is to overdredge bottom and sides of channel to prevent too much movement of dredge and to obviate having to go over work again to clean up. The result is usually an uneven bottom and uncertain slopes.

Q. Is it known by the advocates of the Fruhling dredge that the ordinary American non-propelling suction dredge can at any time fill its pipes with material of the consistency claimed for the Fruhling dredge?

A. In any comparison with other types of dredge I am naturally only concerned with what the various types are capable of doing regularly, not with what they attain accidentally or intermittently. I am quite aware that suction dredges of cutter and other forms have, by burying their suction heads deep down in soft material, filled their pipes with dredgings not only of the consistency of that habitually pumped by the Fruhling dredge but have frequently *plugged the pipes solid.* I have been by when the dredge crews were engaged in digging the material out again, the job lasting a week. I have also observed a floating pipe line and pontoons bottom up from the same cause. These results are scarcely commercially satisfactory and did not therefore engage my attention. If there is one feature in the Fruhling dredge more valuable than another it is just its ability to pump continuously, and without risk of plugging the pipes, a consistency of material hitherto unapproached. It has also the power under dredge operator's control to instantly increase or decrease the water included in dredged material to suit rapidly changing conditions.

Q. Is it known by the Fruhling advocates that although possessing this power we have found that commercially we can handle more material to greater distance by limiting the percentage of solid to 15% or 20%? This occurs because: (a) The current runs proportionately faster and about catches up with the sum total in yardage; (b) After issuance from the pipe it does not heap up at the pipe end but spreads out to a reasonable grade by the very means of the surplus water.

ABSTRACT FROM RECORDS OF U. S. E. DREDGE "GEDNEY."

Material.	Sticky mud and fine sand.	Mud, sand and shell.	Coarse sand.	Sand.	Sand, mud and cobble.	Sand and mud.	Sand.	Average, New York Harbor.
Hopper contents, cu. yds.	536	623	605	596	587	562	587	575
Filled, mins.	117	66	46	60	41	60	34	80
Cu. yds. per hr.	276	570	800	595	864	551	1,047	436
Cu. yds. per min.	4.6	9.5	13.3	9.9	14.3	9.2	17.2	7.2
Speed loaded	7 Knots.							
Dredging depth, ft.	28-35	23-31	28-35	26-35	18-30	18-32	14-26	
Quantity removed, cu. yds.	1,000,000	250,000	500,000	800,000	189,000	225,000	421,000	2,754,902
Cost per cu. yd., cents	17.0	8.6	7.2	7.4	10.3 (a)	10.6	5.1 (b)	11.7 (c)

Cost per cu. yd. includes all expenditures chargeable against the ship, except office expenses (superintendence is included), investment and amortization.

(a) Early stages of work were expensive on account of circuitous course to dump. Last month's work at this locality cost 6.6 cts.

(b) Maximum day's work here at rate of 24.8 cu. yds. per min. Maximum for best month, 18.8 cu. yds: Cost includes expense of taking ship from New York to Gulf of Mexico and bringing her back to New York.

(c) Cost includes maintenance and care of ship during periods of idleness, due to lack of appropriations, delays of contractors and much time necessarily lost while awaiting decisions from War Department.

A. By a change to the Fruhling type of pump from that Mr. Wood uses, which is no doubt a very good water thrower, he would be able to considerably increase his percentage of solids without perceptibly decreasing speed of flow. This, too, with an actual reduction in power expended. It is no doubt very satisfactory that the tremendous gush of water carrying the dredgings, 80 to 85% of the whole, can be usefully employed in grading, since that is all that can be got from the money spent in handling it. However, I have yet to be convinced that 40 to 50% of water would not be equally satisfactory for this purpose. Incidentally such a reduction in the surplus water might obviate great losses of material carried away through the overflows and runways which is so troublesome a feature in filling jobs by usual methods. Mr. Wood will be familiar with the recent accident at Fort Crockett where the enormous amount of water pumped in with the filling by one of his dredges undermined and burst through a massive stone revetment with very serious consequences.

Q. Is it not known by the Fruhling advocates that the propelling type of dredge has been studiously avoided by our private contractors because of defects inherent in the type and which have bankrupted most of those who have undertaken the business? The defects are: (a) The draft is so great that the dredge cannot compete for the vast proportion of shallow dredging in our bays and inland channels. (b) The first cost is prohibitive. (c) The complications of self-propelling craft in connection with maritime laws and certified officers add much to the expense without affording special dredging skill.

A. That the hopper suction dredge should be barred by contractors because its use has bankrupted some individuals who have employed it surely indicates faulty logic. For every such case it is possible to point out a corresponding one in which the contractors have assigned after using the very best and latest appliances of the type Mr. Wood advocates. As a matter of fact the value of any appliances is not in the slightest degree affected by the bad judgment and incompetence of its users who come to grief. No tool, however perfect, is proof against such a handicap. To study the proposition well beforehand, to get the very latest and most perfect appliances and to work them to their utmost capacity under first-class superintendence has always seemed to me the essential characteristics which have led to the great successes in American contracting. That dredging should be any exception I take leave to doubt.

In discussing the defects in the hopper suction dredge it is not altogether surprising that your correspondent has ignored those really inherent in favor of others which are merely incidental. What objection can be raised to reasonable draft and to a navigating staff for dredges which are specially designed to work in channels in the open sea and in the great harbor and river channels such as those at New York, in the Delaware or the Mississippi Passes? Let me ask Mr. Wood how much work his finest cutter head dredge would do, say on the Ambrose channel, before it was pounded to splints.

The one inherent defect in all dredges is their inability to do work under all and any sort of conditions and it will not take long to convince a contractor with several million yards of material to dredge and deposit at a distance in open water that his makeshift appliances, relics of some inland water contract, are the very worst sort of plant for such an undertaking.

The scope of the Fruhling dredge is assured in the United States and elsewhere when it is realized that it is better to build a special tool for special work and to specialize on that work, than to endeavor with cheaper appliances to cover indifferently a seemingly wider scope. Its field does not depend on any future Galveston or similar contract—plenty of work lies ready at hand in ordinary maintenance work in every maritime state in the Union.

As regards the dredges employed by Messrs. Goedhart and Bates at Galveston, these are Dutch built and designed. I am familiar with their system of repumping hopper contents, but as it is not an American product I did not refer to it. It is neither simpler nor better than the Fruhling method and introduces a large and uncontrolled percentage of outside water which is precisely what we wish to avoid.

Mr. D. F. Haasis' letter, with its valuable data on the attainments of the hopper suction dredge "Gedney," is of particular interest. This vessel has come especially under my notice and I have the more pleasure therefore in comparing her qualities with those of a similar vessel of the Fruhling type. Considering the antediluvian design of the "Gedney" and her obsolete pumping outfit, the results given by Mr. Haasis are distinctly creditable. It is quite evident, however, that in dredging in mud she suffers from all the defects inherent in dredges of her type. From Mr. Haasis' description of the chunks in the discharge it is evident that "gulping" takes place—the mud temporarily choking the pipes near the pump until sufficient vacuum is created to break the mass and to suck it into the pump where it is chopped or balled by the impeller. So long as the suction pipes are not actually plugged, this action is not necessarily detrimental; but the great amount of water lifted causing a heavy loss by sluicing of the dredgings through the hopper overflow is a serious matter.

The writer cannot admit that it is of little importance whether the mud dredged remains in the hopper or not. Mr. Haasis considers that "the lost material is carried away by the tide and settles on the adjacent shoals." It might just as reasonably be inferred that it would find its way back into the channel bed and be pumped over and over again. The following extract from Col. Marshall's report for 1907 touches on this point:

Throughout the entire length of the Main Ship Channel, about 5 miles, the currents run crosswise to the channel and carry over the channel edge mud from the shoals on either side. The frequent passage of deep ships prevents this mud from settling permanently in the middle of the channel and confines it to the edges, where at present rates it is depositing faster than the "Gedney" can remove it.

By Mr. Haasis' figures it takes an average of two hours to load the "Gedney" with 536 cu. yds. of mud presumably from the Main Ship Channel to New York Harbor. Allowing another 1½ hours for going to dump and returning, this gives 3½ hours occupied for each load. Assuming it were possible to fit the "Gedney" with Fruhling gear, the time to fill the hoppers would not exceed ¼ hour. With the same allowance for dumping this gives 1¾ hours for each load. In fact this figure would be reduced, for no Fruhling dredge of the "Gedney's" size would be built without such improvements in propelling and pumping machinery as would result in considerable economy of upkeep. In any case the field cost would be cut right in half.

In reply to Mr. Haasis' enquiry as to the depth at which the Fruhling "Dredge VII." operated on trials, it may be stated that the results given in Table I. in my article were obtained with a dredging depth of 25 to 40 ft. On various other occasions depths have varied from 20 to 70 ft. In Table II. of my article the results for the dredge "Fruhling" were obtained at a depth of 30 ft.

I cannot accept as accurate Mr. Haasis' method of comparing place with scow measure. In the present case he has certainly vitiated his estimate for "Dredge VII." by using figures given for working in soft mud in obtaining his place measure, while for the amount secured in the hopper he has taken 33 cu. yds. per min., which is expressly stated to be an average result in sand over a long period of time. If this method is to be used at all, it certainly should not be for sand where the Fruhling head does not sink deeply, but scrapes and rakes up the surface ahead the drag.

We have only admiration for the skill and ingenuity which have enabled this antiquated craft to deal at all with the serious task on which she is engaged. The nature of the work, practically in the open sea, the amount and class of the material to be lifted and the long trip to a dumping ground call for equipment of more up-to-date type.

I am Sir,
Very truly yours,
John Reid.

41 Park Row, New York City, March 30, 1908.

EXPERIMENTAL WORK AT THE GATUN DAM SITE.*

By CALEB MILLS SAVILLE,† M. Am. Soc. C. E.

Experimental work on a large scale is now under way at a station which has been established at Gatun for the purpose of investigating the various soils and rocks which will be used in the construction of the dams and also the materials that are found in the foundations and in the portions of the ground underlying the embankments and walls to be constructed. It is expected that these experiments will show clearly the physical characteristics of the materials themselves, and demonstrate their condition when placed in embankments or when forming foundations subjected to excessive loads or heavy water pressure.

The principal investigations are:

1. Experiments with soils in a wooden tank to ascertain the slope of the water in the embankment; the amount passing through the soil; the best method of depositing soil in order to make it resist the passage of water; and the effect of a load on the embankments.
2. Experiments to determine the frictional resistance of soils and rocks to the passage of water.
3. Experiments to determine the frictional resistance of soils to pressure.
4. Wash drill and diamond drill borings.
5. Test pits in the dam location on Gatun Island, the lower part of the Chagres Valley, and on the hill at the spillway.
6. Experiments to determine the action of the materials under heads of water similar to that expected in the dam.
7. Determination of the best mixture of available materials for making impervious embankments and the action of these mixtures under pressure and when exposed to the action of water.

Beside the borings and soundings that are being made to investigate the material in the foundations of the dam, other investigations are under way for the purpose of developing the location and extent of the best materials from which to construct the big embankment. Thus far the returns are of a most satisfactory character, both as to quality and quantity.

(1) Experiments are being made in a water-tight wooden tank, 80 ft. long, 6 ft. wide and 8 ft. high. In this a section of a dam is being constructed one-twelfth the size of the actual structure. Into the tank materials are being pumped under conditions similar to those that will exist in the dam itself. When completed, the experimental section will be subjected for a considerable period to water pressure and careful records kept of the rate at which water may percolate into the material. These observations will be made by means of glass gages set up outside the tank at distances of five feet apart, and connected with perforated horizontal pipes with the interior of the embankment. In this way the slope of saturation through the embankment can be determined and the actual dam itself so designed that no water of percolation can find its way to the outer slope until it has lost all its pressure and power of damage. Several dams of various materials will probably be constructed in this tank, from time to time, in order to determine the best materials for use in the actual work.

(2) Experiments are being made in galvanized iron tanks about 3 ft. in diameter and 5 ft. high. In these a layer of the material to be investigated is placed and exposed to a constant head of water. Careful observations by glass gages make it possible to determine the quantity of water that will pass through under various heads of water, and the thickness of material that will be necessary to make the embankment practically impervious.

Studies of the land slides at various points on the Isthmus are being made and experiments made with large size sliding boxes to determine the friction angle and the coefficient of friction for the various materials.

The wash drill and diamond drill investigations are on a much more extensive scale than previously undertaken, and the ground is being so thoroughly explored, and to such great depth, that it is felt that no conditions can obtain that will materially change the plans after they have once been decided upon.

On Gatun Island, at a point about half way between the old channel of the Chagres River and the French canal, an exploration pit is being excavated, which it is proposed to carry down a distance of at least 100 ft. This pit is 20 ft. square, and has now reached a depth of 18 ft. As yet the materials excavated have been compact clay and sand, containing very little water, a hand pump, intermittently worked, being sufficient to drain the pit. The excavation is being carefully watched, and it is planned every 10 ft. and oftener if the material changes, to make mechanical analyses and filtration determinations.

In order to determine the action of the soil and rocks when exposed to heavy water pressure, cylinders of the materials to be investigated are enveloped in strong iron tubes, varying in size from 3 ins. to 8 ins. in diameter, and then the specimen is subjected to water pressure, varying from 20 lbs. to 80 lbs. per sq. in., corresponding respectively to 46 ft. and 185 ft. head. Abrasion tests are being made of the several rocks found, and also experiments to determine the resisting qualities of the rocks when exposed to the eroding action of a stream of water under pressure.

Beside the ordinary information obtainable from wash and diamond drill exploration regarding the character of the underlying strata, investigations are also being made to determine the velocity of flow of the underground water. These determinations are being made by the electrical method, devised and successfully used by Professor Schlicter on the U. S. Geological Survey work. The underground water conditions are also being investigated by seepage and pumping tests.

*From the "Canal Record" of April 8.
†Asst. Div. Engr., Gatun Dam Division, Isthmian Canal.

A CONCRETE TRESTLE is a somewhat novel type of structure built across the Salt River, in Illinois, by the Chicago, Burlington & Quincy Ry. The structure is 477 ft. long, with six-pile bents spaced 14 ft. c. to c., and with an average height of 10 ft. The piles are of reinforced concrete, 22 ft. long, and were driven by a pile-driver with a 3,000-lb. hammer falling 24 ft. The piles are all vertical and are capped by deep concrete cross girders supporting the slabs which form the floor or deck. The trestle was built by the railway company's forces, under the direction of Mr. T. E. Calvert, Chief Engineer, and Mr. C. H. Cartlidge, Bridge Engineer. A somewhat similar structure is a promenade on the sea front at Long Beach, N. Y. This has the outer piles inclined as batter piles, with the cap girders extending beyond them. The piles were put down in the sand by the aid of water jets. This structure is about five miles long.

PROGRESS AND COST DATA, U. S. RECLAMATION SERVICE PROJECTS.

The monthly bulletin of the U. S. Reclamation Service for April, 1908, contains a number of interesting items detailing the progress of work on the different projects, expressed in percentage of total work completed, and also detailing the cost of several interesting pieces of work. Some of these we have abstracted as follows:

PROGRESS ON VARIOUS PROJECTS OF THE U. S. RECLAMATION SERVICE TO MARCH 31, 1908.

	Per cent. completed.
Belle Fourche Project as a whole	46.5
Orman & Crook, Belle Fourche Dam (suspended)	24
Bufford-Trenton Project as a whole	36
Carlsbad Project as a whole	96
Avalon Dam	99.5
Garden City Project as a whole	83
Huntley Project as a whole	99
Klamath Project as a whole	30
Lake Tahoe Storage	12.5
Lower Yellowstone Project as a whole	79
Pacific Coast Construction Co., Lower Yellowstone Dam	40.5
North Platte Project as a whole	77.5
Geddis & Seerie Stone Co., Pathfinder Dam (revised estimate)	58
S. R. H. Robinson & Son, Diversion Dam and head works	58
Okanogan Project as a whole	75.5
Storage Works	55
Payette-Boise Project as a whole	93
Utah Fireproofing Co., Boise Dam	88
Upper Deer Flat embankment, force account	85
Rio Grande Project, Leasburg division	98
St. Mary Project as a whole	18
Salt River Project as a whole	67
J. M. O'Rourke & Co., Roosevelt Dam	34
Granite Reef Dam	80
Shoshone Project as a whole	38
U. S. Fidelity & Guaranty Co., Shoshone Dam	22
Strawberry Valley Project as a whole	18.5
Strawberry tunnel	7
Sunnyside Project as a whole	20
Sun River Project as a whole	5.5
Tieton Project as a whole	26.5
Truckee-Carson Project as a whole	90
Umatilla Project as a whole	70.5
Cold Springs Dam	86
Uncompahgre Valley Project as a whole	58
Gunnison tunnel	69
Williston Project as a whole	45
Yuma Project as a whole	45
Laguna Dam, force account (J. G. White contract)	71

SHOSHONE DAM.—The Shoshone Dam, now under construction near Cody, Wyo., will have a total height, from the lowest foundation to the parapet wall, of 320 ft. This will be somewhat in excess of the estimated height, owing to the fact that bed rock was found to be at a lower depth than was supposed. The excavation to bed rock has been completed and the first concrete masonry was placed on March 31, 1908.

PATHFINDER DAM.—The Pathfinder Dam, on the North Platte River in Wyoming, had reached an average height of 81 ft. above the foundation, or 129 ft. below the roadway over the crest, at the close of March, 1908. It is estimated that the dam will be completed by the end of the present year. The high-pressure gates (see Eng. News, Jan. 2, 1908) have been installed, with the exception of the electrically operating machinery.

GUNNISON TUNNEL, COLORADO.—The cost of concrete floor lining in this tunnel for the month of September, 1907, is reported as shown by the accompanying tab'e.

COST OF CONCRETE FLOOR LINING, GUNNISON TUNNEL.

Distribution of cost.	Total cost.	Unit cost per cu. yd.	Unit cost per lin. ft.
Preparing floor for lining:			
Superintendence	$34.00	$0.20	$0.05
Handling material	118.56	0.71	0.16
Hauling material	33.60	0.20	0.04
Total	$186.16	$1.11	$0.25
Placing floor lining:			
Superintendence	$36.25	$0.21	$0.05
Mixing & hauling concrete	92.00	0.55	0.12
Placing grade boards	31.92	0.19	0.04
Placing concrete	101.84	0.61	0.14
Cement	756.00	4.50	1.01
Sand and gravel	148.68	0.88	0.20
Total	$1,166.69	$6.94	$1.56
Grand total	$1,352.85	$8.05	$1.81

The tunnel is 10 ft. wide on the bottom and the average floor thickness is about 0.6-ft. In making the concrete, 6 sacks of cement, 12 cu. ft. of sand and 26 cu. ft. of gravel were used to each cubic yard of concrete. For the 749 lin. ft. of floor lining to which the cost data in the foregoing table applies, 252 bbls. of cement and 236 cu. yds. of aggregates were used. The rates of pay for eight hours labor were as follows: Concrete foremen, $5; general foreman, $4; laborers, $3.04; motormen, $3.04; brakemen, $2.56. Up to the end of March a total of 13,000 lin. ft. of concrete floor and about 8,500 lin. ft. of concrete side walls and arch had been placed in the west portal of the tunnel. This tunnel forms a part of the Uncompahgre Project.

REINFORCED-CONCRETE CANAL LINING.—The cost of reinforced-concrete canal lining for 7,500 ft. of the South Canal of the Uncompahgre Project (Colorado) of the U. S. Reclamation Service is given in the bulletin. The lining was 6 ins. thick. The work and its cost are described as follows:

The cost data given below relate to the lining of 7,500 lin. ft. of South Canal, Uncompahgre Project, Colo., with a 6-in. layer of reinforced concrete. Two sections of the canal are considered, each having side slopes of ½ to 1, one having a bottom width of 8 ft. and a vertical lined depth of 10.5 ft., and the other having a bottom width of 13 ft. and a vertical lined depth of 8.4. Each of these sections, it will be noted, requires about 0.6 cu. yd. of concrete per lin. ft. of canal. The total amount of reinforcement placed is 245,085 lin. ft. of ⅜-in. corrugated steel bars, weighing as a whole about 119,140 lbs. The steel was hauled an average distance of 4 miles from the railroad siding to the canal work. One man was employed at placing the steel in the concrete during construction. Labor was paid for at the rate of $0.25 an hour, and teams with drivers at the rate of $0.45 an hour. The cost of superintendence of the hauling and placing of the steel has been included with those items and amounts in each case to 15% of the original costs thereof.

COST OF REINFORCING STEEL IN SOUTH CANAL LINING.

Distribution of cost.	Unit cost per lb.	Unit cost per lin. ft. of bars.	Unit cost per cu. yd. of concrete.
Cost at factory	0.0245	0.0119	0.648
Cost of freight	0.0043	0.0021	0.114
Cost of hauling	0.0007	0.0003	0.018
Cost of placing	0.0043	0.0021	0.115
Totals	0.0338	0.0164	0.895

CEMENT MILL, ROOSEVELT, ARIZ.—This cement mill, operated by the U. S. Reclamation Service to supply cement for the construction of the Roosevelt Dam, produced 10,614 bbls. of cement in February, at a unit cost of $1.81 per bbl. In comparing this cost with the price at which cement could be purchased, we may add, it should be borne in mind that there should be added to the latter the cost of freight to the nearest railroad station and the cost of the long team haul to the dam.

VICTIMS OF THE CHELSEA FIRE, near Boston, Mass., now number 12 according to latest reports at the time of going to press. The essential features of this catastrophe were outlined in our issue of April 16.

FORTY JAPANESE WERE KILLED IN A SNOW-slide, near Albert Canyon, west of Calgary, Alberta, during the week of April 18, on the Canadian Pacific R. R., according to current press reports. This is said to have been one of the worst slides in the region and swept away two boarding cars containing these Japanese.

IN A REAR-END COLLISION of two excursion trains about seven miles from Melbourne, Australia, April 19, 41 persons are reported to have been killed and 60 injured. Many of the injured were caught in the wreckage and were released with some difficulty as the wreckage took fire and was almost completely burned.

HEAVY FLOODS THROUGHOUT TEXAS and southern Arkansas, resulting from long-continued and severe rains during the past week, have caused an immense amount of damage to property and probable loss of life, although the extent of the damage cannot be definitely ascertained until the waters subside. The Brazos, the Trinity and the Texas Colorado Rivers, especially, have been rising steadily until in most places they are now higher than at any time during the past 30 years. In many of the smaller towns the residents have been driven from their houses, but the greatest damage has been done at Fort Worth and at Dallas, in both of which cities large portions of the lower parts of the town are under water and many families have been rendered homeless.

IN A BOILER EXPLOSION on April 19, at an ice plant in Tipton, Ind., heavy pieces of machinery, timbers and bricks were blown through the walls and roofs of nearby residences causing a total damage of about $25,000. The engineer was blown some distance from the plant but was not injured. He claims that he was carrying normal steam pressure and amount of water. A vertical boiler in a small pumping station of the Cleveland, Cincinnati, Chicago & St. Louis Ry., exploded on April 13, killing the attendant. The boiler bottom was lifted over the trees and rammed into an embankment of the Baltimore & Ohio R. R., 300 ft. away.

A COAL MINE WAS DYNAMITED at Durango, Colo., April 14, following an anonymous warning to the Superintendent of the Royal Coal & Coke Co., to have the workings made safe. An inspection of this mine, the Champion, was made after the receipt of the letter and the workings were declared safe. It is reported that by the explosion two Italians were seriously injured and some workings were wrecked.

TWO DERAILMENTS ON ELEVATED RAILWAYS in Chicago have occurred since the one noted in our issue of April 9. The first of these was the derailment of a train of the Metropolitan Elevated Ry. on the curve connecting the terminal loop with the company's own line. It is thought to have been caused by lumber or other material lying near the rail, as repair material is stored and used near this junction. The second accident occurred April 17, on the South Side Elevated Ry. near 39th St., and is said to have been caused by the breaking of a turnbuckle on a truss rod, allowing the rod to fall upon the track. In neither case did any serious injury result, although the passengers in the derailed cars received a severe shaking.

THE COLLISION OF A FOUR-CAR ELECTRIC TRAIN on the Bath Beach line of the Brooklyn Rapid Transit Co., with a trolley car at Eighty-sixth St. and Eighteenth Ave., Bath Beach, Brooklyn Borough, New York City, caused the injury of 13 persons on April 20. The trolley car was squarely across the tracks of the elevated line, which runs on the surface at this point, when struck by the train. On May 21, 1907, about 50 persons were injured in a similar accident at the same spot, and since then a flagman has been stationed there. Signals are also displayed on all four tracks leading to the crossing and it is supposed that as the motorman on the trolley car was a green hand he misinterpreted the signals in some way. The wreck was fired by a short circuit of the overhead system, but by the efforts of police and firemen the flames were checked until all the victims were released. The motorman of the car and train and the flagman were arrested and held pending an investigation.

IN A HEAD-ON COLLISION of two heavy interurban cars on the Washington, Baltimore & Annapolis Ry., April 14, about seven miles from Annapolis, Md., five persons were injured. The cause is attributed to a misunderstanding of orders resulting from a change of schedule on April 12. A coal car attached to a dinky and carrying 15 laborers on the Pittsburg, Harmony, Butler & New Castle Street Ry. broke from the locomotive and dashed down the rough track at a steep grade about ten miles from Evans City, Pa. In rounding a curve the workmen were thrown out and against a fence. The car continued into Evans City where it jumped the track and ploughed into a flouring mill.

NINE MEN WERE KILLED in a premature dynamite explosion at Armagh, Canada, about 22 miles south of Quebec on April 18. The men were working in a rock cut near the construction camp of Murdock Bros. on the Transcontinental Ry. A "separator mill" of the Aetna Powder Co., at Miller, Ind., was completely demolished April 13 by an explosion of nitroglycerine. The buildings were vacant at the time.

THE METROPOLITAN LIFE BUILDING TOWER, now in process of construction at Madison Square in New York City, and which is already the highest building in the world outside of the Eiffel Tower at Paris, is to have 37 ft. added to its height, thus bringing the total number of stories, including two in the basements, up to 50, and the top of the surmounting glass ball 693 ft. above the sidewalk level. As noted in our issues of Jan. 10, 1907, p. 49, and Jan. 31, 1907, p. 139, this tower is patterned after the Campanile at Venice, a rectangular section to a point 492 ft. above the ground, topped by a pyramidal structure 94 ft. high and finished with a tapering octagonal tower. The additional height is to be gained in the final octagonal tower, which not only is to be treated in a more ornate manner than as first designed, but is to contain a large electrically-lighted lantern, which should be visible at night for many miles around the city. The new design has an observation balcony 662 ft. above the sidewalk, or four feet above the very top of the structure as first projected, and another enclosed observation room two stories lower, for the benefit of those who do not care to risk the dizzy eminence of the unenclosed upper balcony. The revised design has been prepared in the office of the architects of the building, Messrs. N. Le Brun & Sons.

THE MUNICIPAL WATER SUPPLIES OF KANSAS were summarized as regards source, purification, ownership and the use of meters, in a paper read not long ago by Prof. William C. Hoad, Sanitary and Civil Engineer of the Kansas State Board of Health, before a conference of health officers. A total of 36 cities, with a population of 293,000, take their supply from rivers and small streams; 9 cities, with a population of 25,000, draw water from lakes and ponds; 50 cities, with a

population of 208,000, use shallow wells; and 22 cities, with a population of 73,000, depend on deep wells. The populations given are "from the assessors' census of 1907." As here used, shallow wells include those of depths of not over 100 ft. Of the 44 cities depending on surface supplies, 17, with a population of 61,000, use no method of purification for the water; 6 cities, with a population of 50,000, purify their water supplies by plain sedimentation; 3 cities, with a population of 59,000, adopt the method of sedimentation with coagulant; 5 cities, with a population of 9,000, use a process of straining through coarse material; and the remaining 10 cities, having a population of 137,000, filter their water supplies. Of 117 water-works, 89, serving a population of about 316,000, are owned by municipalities, and 28, serving about 288,000 people, are owned by companies. Of 81 cities reporting definitely on the use of meters, 41 sell by meter one-half or more of the total quantity furnished, "and of this number 34 are municipal and 7 are private plants." The 81 plants have an average combined daily delivery of over 20,000,000 gals., of which 53% is metered.

THE FLOODS ON THE UPPER MISSOURI RIVER which caused the failure of the Hauser Lake steel dam, as noted in our last week's issue, have passed down the river without causing the immense damage first anticipated. In addition to the destruction of the dam the greatest loss of property occurred at Craig, Mont., a small town about 140 miles north of Helena. Here the entire town was destroyed and the residents, about 100 in number, were forced to take refuge in the hills. At Great Falls, the site of the Boston & Montana smelter, the rising waters were diverted from the works by a hastily constructed wing dam, and held in check until the floods had subsided. It was reported in the daily newspapers that the permanent dam at Great Falls had been dynamited to allow the free passage of the flood down the river, but this statement is incorrect; the dam at great Falls was not destroyed either by the floods or by intentional explosion. Investigations have not as yet ascertained the exact reason for the failure of the Hauser Lake steel dam. So far as can be learned the middle 300 ft. of the structure was washed out, and it is stated that the portion that remains is founded on solid rock, while the destroyed section was founded on gravel, although the steelwork for the two portions is the same.

PRODUCTION OF TURPENTINE FROM YELLOW pine waste has been studied by the Forest Service, through its section on Wood Chemistry. It is found that for the recovery of turpentine from waste wood the steam distillation process is far superior to destructive distillation, making a more uniform crude turpentine and usually a higher-grade refined product. The Forest Service states that the wastage from the yellow pine cut each year would yield as much turpentine as the entire present annual output in this country, with a value of some $14,000,000. They further assert that, at the present rate of cutting, the supply of long-leaf yellow pine in the South will be practically exhausted in twenty years, but that the methods of exploitation now in use convert only about half the tree into market product. The importance of waste utilization is based on these facts. The study of turpentine production will be continued by the service, in the belief that there is no reason why steam (waste-wood) turpentine should not bring at least as high a price as gum turpentine.

WORK ON THE NEW YORK BARGE CANAL is described in a monthly publication issued by the State Engineer's office at Albany, N. Y., the first number of which appeared on June 1. The progress of work on the portions now under contract to Feb. 1, 1908, is summarized in the following table.

As the total estimated cost of the work is about one hundred million dollars, it will be seen that only a little over one-fifth of the work is as yet under contract.

Contract No.	Contractor.	Total value of work: Put under contract.	Done to Feb. 1, 1908.	Percentage of work done to Feb. 1, 1908.
1.	Empire Engineering Corp.	$605,008	$338,690	56.0
2.	Ferguson Contracting Co.	902,145	468,110	51.8
3.	Sundstrom & Stratton	677,019	528,560	78.0
4.	Empire Engineering Corp.	725,065	214,990	29.7
5.	Empire Engineering Corp.	375,872	125,820	33.5
6.	Frank A. Maselli	1,022,601	497,350	48.6
7.	Groton Bridge Co.	97,537	37,800	38.8
8.	Pittsburg-Eastern Co.	1,434,755	146,390	10.2
10.	Mosier & Summers	1,107,610	103,550	9.4
11.	Fort Orange Construction Co.	1,359,475	235,260	17.3
12.	Stewart-Kerbaugh-Shanley Co.	3,391,716	0	0
14.	Acme Engineering & Cont. Co.	2,935,763	14,460	0.5
15.	Atlantic, Gulf & Pacific Co.	1,520,547	180,260	11.8
16.	United Construction Co.	63,473	0	0
17.	The Scofield Co.	836,643	57,560	6.7
18.	O'Brien & Hoolihan Cont. Co.	861,599	130,490	15.2
19.	Great Lakes Construction Co.	1,000,497	51,190	5.1
25.	Atlantic, Gulf & Pacific Co.	1,717,849	112,320	6.5
27.	Kinser Construction Co.	943,542	64,210	6.6
34.	M. Fitzgerald	22,449	15,340	68.0
35.	Gilmour-Horton-Allen Co.	739,261	3,430	0.5
	Total	$22,340,426	$3,325,780	

TESTS ON REPRESENTATIVE FUELS OF THE UNITED STATES IN THE PRODUCER-GAS POWER-PLANT AT THE FUEL TESTING LABORATORY OF THE TECHNOLOGIC BRANCH, U. S. GEOLOGICAL SURVEY.

Fuel.	Proximate analysis, per cent.: Moisture.	Volatile matter.	Fixed carbon.	Ash.	B. T. U. per lb. of fuel.: As fired.	Dry.	Cu. ft. standard gas produced per pound of equivalent* fuel consumed by producer plant.: As fired.	Dry.	B. T. U. per cu. ft. of standard gas.	Pounds of equivalent* fuel per B. HP.-hr.: As fired.	Dry.
Florida peat	21.00	51.72	22.11	5.17	8,127	10,289	28.5	30.1	175.2	2.57	2.03
Average of four lignites	35.5	28.96	27.72	8.27	7,164	11,038	26.3	40.3	169.9	2.43	1.73
Av. of four Illinois coals	11.51	31.81	43.46	13.22	10,651	12,030	49.6	56.1	153.2	1.66	1.47
Av. of four Penna. coals	3.47	19.68	67.31	9.54	13,651	14,136	71.4	74.0	141.6	1.16	1.12
Av. of four W. Va. coals	2.47	32.12	60.24	5.17	14,248	14,610	77.5	79.5	149.6	1.03	1.00

*Equivalent fuel includes that used in producer, and also the amount required to generate the steam necessary for operating the producer.

TESTS ON REPRESENTATIVE FUELS in the producer-gas power-plant of the fuel testing laboratory of the Technologic Branch, U. S. Geological Survey at St. Louis, Mo., have given the results shown in the accompanying table. The equipment used was a 250-HP. pressure producer with a centrifugal tar extracter and gas holder. A 225-HP., three-cylinder, vertical gas-engine, belted to a generator, produced power which was measured by electric instruments connected with the switchboard. The results shown are better than those from steam plants of a corresponding size. Of the four Pennsylvania coals tested, two came from the lower Kittanning bed, one from the lower Freeport and the fourth from the Pittsburg bed. Of the West Virginia coals, one came from the Ansted bed, another from the Eagle, both of these being mined in the New River district; a third from the Pittsburg and the fourth from the Keystone bed.

CONCRETE GIRDER AND SLAB BRIDGES are being used in some of the track-elevation work in Chicago. On the work of the Chicago, Burlington & Quincy Ry. between Canal St. and Blue Island Ave., rows of concrete posts on the curb lines and in the middle of the street carry transverse girders, supporting the ends of the slabs which form the deck. Some of the first of these structures have steel posts and plate girders to support the slabs. The two roadway spans are 24 ft. 3 ins. c. to c. of posts, with sidewalk spans of 8 ft. 9 ins. from center of post to face of abutment. The posts and girders are built in place. The slabs are built in the yards, transported on flat cars and set in place by a 100-ton wrecking crane. For similar work on the Illinois Central Ry., the posts and girders are placed by walls having arched openings. The floor slabs rest upon these walls and the abutments.

THE STATEMENT THAT A 1,000-FT. SHIP will be built by the White Star Line, which has been widely circulated, appears to lack confirmation. The most definite reports are to the effect that a very large transatlantic liner to be called the "Olympic" will soon be started for the White Star Line. No definite statement appears to have been made, however, as to its length, or as to whether it will exceed the Cunard monsters in size.

THE ORE DOCKS ON THE GREAT LAKES have an aggregate storage capacity of 1,289,723 tons, according to a tabulated list prepared by Mr. W. A. Clark, Chief Engineer of the Duluth & Iron Range Ry. There are 24 of these docks, owned by eight railways, and they are located at six ports. The largest is one of the three owned by the Duluth, Missabe & Northern Ry. at Duluth, Minn.; this has 384 pockets, with a combined capacity of 119,274 tons. This dock is 2,304 ft. long, 57 ft. wide and has its deck or track floor 72 ft. 6 ins. above the water. The Great Northern Ry. has a dock of 100,980 tons storage capacity at Superior, Wis. A table of the ore docks given in an issue of Oct. 26, 1904, showed 23 docks with a capacity of 1,042,966 tons. Some of the older docks, however, have been replaced by larger structures. All the docks noted above are at American ports. There is one Canadian dock at Michipicoten, but its capacity is not given; it is, however, a small structure with only 12 pockets.

TYPHOID FEVER IN CINCINNATI, OHIO, before and after the introduction of filtered water, is discussed in a weekly report of the local Board of Health, dated March 20, 1908 (Mark A. Brown, Health Officer). Although the whole of the city was not supplied with filtered water until about Nov. 1, 1907, yet a gradual improvement in the supply began on April 1 and continued until Nov. 1. At the start, the improvement consisted simply of a less badly polluted water for the high-service section of the city. On Sept. 1 the whole city was supplied with this better class of raw water. Filtration was begun about Oct. 20, and since Nov. 1, 1907, nothing but filtered water has been supplied. The number of cases and deaths from typhoid fever in Cincinnati for the past three years and for the first two months of 1908 have been as follows:

	1905. Cases.	1905. Deaths.	1906. Cases.	1906. Deaths.	1907. Cases.	1907. Deaths.	1908. Cases.	1908. Deaths.
January	98	15	92	22	24	24	17	3
February	67	13	86	19	199	16	21	8
March	74	20	72	17	185	23	..	..
April	48	12	78	17	117	16	..	..
May	48	12	106	11	99	6	..	..
June	46	9	452	31	91	16	..	..
July	60	12	286	31	96	15	..	..
August	46	14	275	34	91	13	..	..
September	55	13	123	17	62	5	..	..
October	55	7	87	14	50	10	..	..
November	38	9	115	11	28	7	..	..
December	111	19	168	15	23	6	..	..
Total	746	155	1,940	239	1,252	157	..	..

From April 1 to Nov. 1, 1907, the period of gradually improving water, there were 606 cases of typhoid fever and 81 deaths, as compared with 1,407 cases and 155 deaths for the corresponding months of 1906. For the four months since the whole city was supplied with filtered water, the cases numbered 89 and the deaths 24, as compared with 693 cases and 66 deaths for the four months November-December, 1906, and January-February, 1907.

FORESTRY WORK ON THE DRAINAGE AREA of the water-works of Oberlin, O., is being conducted jointly by the Water and Sewer Department of the village and the Ohio Experiment Station. During the year 1907, the report states,

several bushels of walnuts have been planted in the valley of the river where it was too rough to plow or there was liability of washing. There are upon the farm several thousand young hickories from 10 to 20 ft. high. The wood from these trees is fast becoming the most valuable of our timber. These trees have all been gone over and any that were forked so as not to produce straight clear growth have been trimmed. There is promise that these trees will be a very valuable asset in a few years.

During the year, 5,200 catalpas, 3,000 green ash and 800 locust trees were planted. The total tree planting to date is as follows: Catalpas, 18,900; mulberries, 1,000; sycamores, 600; locusts, 6,100; tulip poplar, 1,000; white ash, 2,000; green ash, 4,000; black walnut, 500. During the year 1907, $285 was received from wood and saw logs.

TOWN PLANNING IN GREAT BRITAIN is to be put on a better basis than has hitherto prevailed, if the bill introduced into Parliament by the Local Government Board becomes a law. The bill is outlined as follows, in the London "Surveyor" of April 3:

It is proposed that every borough and district council may prepare a town-planning scheme to which all owners of land affected by such scheme will be bound to adhere. In districts where no scheme is prepared the Local Government Board may compel a town plan to be adopted. Local authorities may prepare schemes of town development, which may include the provision of parks and open spaces, either singly or conjointly, and it is to be hoped that in some of the smaller and yet closely-populated districts this latter method will be adopted, otherwise there may still be a lack of far-sighted planning on a wide scale. The bill contains provisions for the formation of a joint committee of two or more local authorities in cases where the Local Government Board think it expedient that such a committee should be constituted. Owners whose property is injuriously affected by reason of a scheme of town planning may claim compensation from the local authority, but the latter may claim against any owner whose property is raised in value and may recover the amount of such increased value.

AGGREGATE LENGTH OF LINES OR PARTS OF LINES ON WHICH THE BLOCK SYSTEM IS IN USE JANUARY 1, 1908.

Automatic block signals.					Nonautomatic block signals.							
Single track.	Double track.	Three track.	Four track	Total.	Single track.	Double track.	Three track.	Four track.	Total.	Total, all kinds.	Total miles operated by company (passenger lines).	Per cent. operated under block system.
4,363.5	5,999.8	197.8	541.9	10,803.0	38,517.0	8,477.6	258.8	652.3	47,875.7	58,678.7	151,455.2	.....

KINDS OF AUTOMATIC SIGNALS IN USE.

Unineloscd disks ("clock-work").		Inclosed disks.		Semaphores. Electro-pneumatic.		Electric motor.		Electro-gas.		Normal	Normal	Total automatic signals.	
Miles of road.	Miles of track.	Miles of road.	Miles of track.	Miles of road.	Miles of track.	Miles of road.	Miles of track.	Miles of road.	Miles of track.	clear miles of track.	danger miles of track.	Miles of road.	Miles of track.
497.6	894.4	1,838.0	3,694.7	416.7	1,333.8	7,143.9	10,685.7	923.1	1,925.5	14,317.1	4,217.0	10,819.3	18,534.1

METHODS AND APPARATUS USED WITH MANUAL BLOCK SYSTEM.

Morse telegraph.		Telephone.		Electric bells.		"Controlled." No track circuit.		Track circuit at stations.		Continuous Track circuit.		Electric train staff.		Block signal stations.
Miles of road.	Miles of track.	Miles of road.	Miles of track.	Miles of road.	Miles of track.	Miles of road.	Miles of track.	Miles of road.	Miles of track.	Miles of road.	Miles of track.	Miles of road.	Miles of track.	Total number.	Closed part time
40,040.3	47,781.3	3,286.8	4,183.3	838.5	1,388.9	5,240.3	5,469.0	726.7	2,118.0	212.0	410.8	234.4	235.6	9,438	2,600

PRACTICES IN THE OPERATION OF MANUAL BLOCK SYSTEMS.

Permissive signaling not allowed.		Permissive signalling allowed.* By three-position signals.		By two-position signal or flag.		By caution card.		Rear-end protection only.		Signals opposite office.	
Miles of road.	Miles of track.	Miles of road.	Miles of track.	Miles of road.	Miles of track.	Miles of road.	Miles of track.	Miles of road.	Miles of track.	Miles of road.	Miles of track.
10,039.9*	12,665.9*	19,531.3	24,989.5	1,788.8	2,214.6	16,567.8	18,846.3	8,130.3	8,220.3	23,196.4	26,830.5

*On some roads where permissive movements are allowed the practice covers only freight service, an absolute block being maintained for passenger trains. The figures 10,039.9 and 12,665.9 should then be increased by 13,851.3 and 17,706.1, respectively, making 33,891.2 miles of road and 30,372.0 miles of track on which permissive signalling is not allowed for passenger traffic.

BLOCK SIGNALING ON RAILWAYS IN THE UNITED States, is the subject of a report just issued by the Interstate Commerce Commission. The report shows the condition on Jan. 1, 1908, and comprises four tables which, taken together, form a complete summary of systems used and show their general features and the rules under which they are operated. These data were collected and compiled by the Block Signal and Train Control Board under the direction of the Interstate Commerce Commission. The accompanying table is a summary of these four tables.

A comparison of the previous report of the Commission on this subject shows an increase of 9,935.5 in block signal mileage since Sept. 30, 1906; 5,959.4 miles of this was of manual and 3,976.1 was of automatic-block installation. The increase in automatic-block mileage was largely on single track lines, being 2,331.1 against 1,645 for two, three and four-track systems.

PERSONALS.

Henry Floy, M. Am. Inst. Elec. E., consulting engineer, has removed his offices in New York City to the City Investing Building.

Dr. Charles Richard Van Hise, President of the University of Wisconsin, will deliver the class-day address at the Michigan College of Mines, Houghton, Mich., on May 1.

Mr. Walter H. Horton, Superintendent of the Rutland Ry., Light & Power Co., of Rutland, Vt., has resigned to become Assistant Superintendent of the Atlantic, Gulf & Pacific Co., of New York City.

Mr. C. F. Hewitt, General Superintendent of the St. Joseph Ry., Light, Heat & Power Co., of St. Joseph, Mo., will succeed Mr. G. C. Pierce as Superintendent of the St. Louis & Suburban Ry. properties.

Mr. James H. Gaylord, M. Am. Inst. E. E., Electrical Assistant to Mr. O. H. Ensign, Consulting Engineer at Madison, Wis., will supervise the installation of apparatus in the new power-house of the Strawberry Valley Reclamation Service project, Utah.

Mr. R. B. Hayes, Superintendent of Construction, U. S. Public Buildings, who has been in charge of the erection of the Custom-House at Baltimore, which is now completed and occupied, has been transferred to the remodeling and extension work on the U. S. Court-House and Post-Office at Rochester, N. Y.

Mr. Guy C. Emerson, M. Am. Soc. C. E., formerly a constructing engineer in the United States Reclamation Service, has been granted a furlough for three years, and has accepted the position of Superintendent of Streets for the City of Boston, in charge of the department recently formed by the consolidation of the Street, Sewer, Sanitary, Street Cleaning and Watering, Bridge and Lamp Departments.

Civil Engineer H. H. Rousseau, U. S. N., Assoc. M. Am. Soc. C. E., of the Isthmian Canal Commission, and Miss Gladys Squiers, eldest daughter of the American Minister, Herbert G. Squiers, were quietly married April 4. The civil ceremony was performed in the presence of a few friends at the Hotel Tivoli, Panama, by Judge H. A. Gudger, Associate Justice of the Supreme Court of the Canal Zone; the church celebration taking place later in the Roman Catholic church.

Mr. J. E. Greiner, M. Am. Soc. C. E., prominent in bridge construction since 1885, has tendered his resignation as Assistant Chief Engineer of the Baltimore & Ohio system, to take effect May 1. Mr. Greiner will hereafter devote his time to private practice as a consulting engineer, in which capacity he has been retained by the Baltimore & Ohio as well as by the Chicago, Burlington & Quincy railways. Mr. Greiner will have offices in Baltimore, New York and Chicago.

Obituary.

Edward R. Lowe, a noted submarine engineer and contractor, who for many years had been the manager of the Lowe Wrecking & Diving Co., died April 16, at his home in Brooklyn, in his sixtieth year. He built the foundation of the old Croton dam, and during the Civil War prosecuted diving operations in Mobile Bay.

Hall Wilson Watts, President of the Railway Club of Pittsburg and Master Car Builder for the Monongahela Connecting Railway, died April 14, at the Allegheny General Hospital in Pittsburg. His demise is presumed to be the result of injuries received by a fall last summer, and from which he never recovered. He was 57 years old.

ENGINEERING SOCIETIES.

COMING MEETINGS.

AMERICAN ELECTROCHEMICAL SOCIETY.
April 30-May 2. Annual meeting at Albany, N. Y. Secy., Joseph W. Richards, Lehigh University, South Bethlehem, Pa.

AMERICAN WATER-WORKS ASSOCIATION.
May 11-16. Annual meeting at Washington, D. C. Secy., John M. Diven, 14 George St., Charleston, S. C.

NATIONAL ELECTRIC LIGHT ASSOCIATION.
May 19-22. Annual convention at Chicago. Secy., W. W. Freeman, 29 West 39th St., New York City.

NATIONAL FIRE PROTECTION ASSOCIATION.
May 26-28. Annual meeting at Chicago. Secy., W. H. Merrill, Jr., 382 Ohio St., Chicago.

AMERICAN FOUNDRYMEN'S ASSOCIATION.
June 9-11. Annual convention at Toronto, Ont. Secy., Richard Moldenke, Watchung, N. J.

MASTER CAR BUILDERS' ASSOCIATION.
June 17-19. Annual convention at Atlantic City, N. J. Secy., Jos. W. Taylor, 390 Old Colony Bldg., Chicago.

AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.
June 22-24. Annual convention at Atlantic City, N. J. Secy., Jos. W. Taylor, 390 Old Colony Bldg., Chicago.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
June 23-26. Annual convention at Denver, Colo. Secy., C. W. Hunt, 220 West 57th St., New York City.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
June 23-26. Annual convention at Detroit, Mich. Secy., C. W. Rice, 29 West 39th St., New York City.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION.
June 23-26. Annual meeting at Detroit, Mich. (In connection with convention of American Society of Mechanical Engineers.) Secy., A. L. Williston, Pratt Institute, Brooklyn, N. Y.

AMERICAN SOCIETY FOR TESTING MATERIALS.
June 23-27. Annual meeting at Atlantic City, N. J. Secy., Edgar Marburg, University of Pennsylvania, Philadelphia.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.
June 29-July 2. Annual convention at Atlantic City, N. J. Secy., Ralph W. Pope, 33 West 39th St., N. Y.

NATIONAL SOCIETY FOR THE PROMOTION OF INDUSTRIAL EDUCATION.—At a recent meeting of the Board of Managers plans were perfected for establishing branch societies in 38 states. The society has established permanent quarters at 546 Fifth Ave., New York City.

CANADIAN SOCIETY OF FOREST ENGINEERS.—A society with the above name has been organized to include all foresters in the Dominion. The following officers have been elected: President, Dr. B. E. Fernow, Toronto; Vice-President, E. H. Campbell, Ottawa; Secretary-Treasurer, F. W. H. Jacombe, Ottawa.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.—The following papers have been announced for the convention at Detroit, June 23-26: "A Method of Cleaning Gas Conduits," by W. D. Mount; "A Method of Checking Conical Pistons for Stress," by Prof. George H. Shepard; "Clutches," with special reference to automobile clutches, by H. Souther; "Horse-Power, Friction Losses, and Efficiencies of Gas and Oil Engines," by Prof. L. S. Marks; "Some Pitot Tube Studies," by Prof. W. D. Gregory; "The Thermal Properties of Superheated Steam," by Prof. R. C. H. Heck; "A Journal Friction Measuring Machine," by Henry Hess; "A By-Product Coke Oven," by W. H. Blauvelt; "Tests of Some High Speed Steam Engines," by F. W. Dean. There will be a symposium upon machinery for conveying materials, with papers by several authorities. The Society for the Promotion of Engineering Education and the Society of Automobile Engineers will also hold their annual meetings in Detroit at this time, which will enable members of each society to participate in the sessions of the other.

APPALACHIAN ENGINEERING ASSOCIATION.—This association will meet in the Assembly Hall of Skinner's Tavern, Fairmont, W. Va., on Saturday, May 2. There will be two sessions, one at 2.30 p. m. and one at 7.30 p. m. The following papers will be presented for discussion: "Practical Phases of Centrifugal Mine Pumps," Mr. O. K. Baker, of the Freeborn Engineering Co., of Kansas City, Mo.; "Electrical Distribution in Shops from Central Stations," Prof. C. R. Jones, West Virginia University; "Common Faults in Concrete Construction," Mr. A. Wagner, Chief Engr. of the Newport News (Va.) Light & Water Co.; "The Upper Freeport Coal Bed," Mr. Ray V. Hennen, Asst. Geologist, West Virginia Geological Survey; "The Failure of a Concrete Bridge and the Action of the Embankment Abutting It," Mr. H. S. Moreland, Resident Engineer, Rochester, Syracuse & Eastern Ry., Clyde, N. Y.; "The Relation of the Locating Engineer to a Modern, Low-Grade, Trunk Line," Mr. G. D. Brooke, Asst. Engr., B. & O. R. R., Morgantown, W. Va.; "The Electrification of Steam Railroads," Prof. Wm. E. Dickinson, of West Virginia University. Messrs. J. A. Holmes, of the U. S. Geological Survey; Wm. N. Page, of the Virginia Ry., and I. C. White, State Geologist of West Virginia, among others, have promised to attend and will address the association.

WESTERN SOCIETY OF ENGINEERS.—At the meeting on April 1, a paper on "Reinforced-Concrete Structures in the Hudson River Tunnel" was read by Mr. Wm. M. Torrance. It related mainly to the terminal work of the tunnels of the Hudson Co., where the tubes are of varying section (increasing towards the station), and are side by side in some places, while in other places they are superimposed, one tube crossing above the other. It was found impracticable to perform this part of the work by the shield system, and the special method adopted was to build the crossing of the tubes as a reinforced-concrete structure. This was built above ground in a steel caisson, and sunk to place by the pneumatic system. By this method the difference of level of the two tubes is reduced to a minimum, the floor of the upper tube forming the roof of the lower tube. The construction of this complicated work was described in some detail, and was illustrated by numerous lantern slides of photographs and drawings.

At the meeting of the Electrical Section on April 10, Mr. T. C. Kelsey presented a paper on "The Relation Between the Banker and the Engineer."

At the meeting on April 15, Prof. A. N. Talbot, of the University of Illinois, read a paper summarizing the results of a series of tests on the strength of culvert pipes made of cast-iron and of reinforced-concrete. The tests were made to indicate the strength of pipes in railway embankments, and the test pipes were embedded in sand in a heavy box fitted with hydraulic cylinders operating beams which transmitted the pressure to a saddle resting upon the sand filling. The relation of different methods of bedding the pipe (on a flat or a concave surface) and of backfilling to the strength of the pipe was discussed. The backfilling not only affects the strength by its distribution of the direct load on top of the pipe, but also affects it by the degree of firmness or looseness with which it is packed against the sides. A number of lantern slides of views and diagrams were given. The paper was followed by considerable discussion, in which Mr. B. J. Ashley expressed surprise that so little had been said in the paper on the discussion as to the relation of thickness to diameter in concrete pipe, or the relation of the concrete mixture to the strength of the pipe. Some of the concrete pipes tested were furnished by five different railways, and the use of such pipe on the Illinois Central Ry. and the Chicago, Milwaukee & St. Paul Ry. was discussed by Mr. A. S. Baldwin and Mr. C. F. Loweth, respectively. The former also referred to the practice of his road in the laying of cast-iron pipe culverts.